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Addis Ababa Science & Technology University
University for the Industry

THESIS ON ANALYSIS AND DESIGN OF OVERHEAD TANK FROM WIND LOAD

IN PARTIAL FULLFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE (M.Sc.) IN STRUCTURAL ENGINEERING

BY

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Declaration

I, the undersigned, declare that this thesis is my Original work and all sources of materials used for the thesis have been duly acknowledged.

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ABSTRACT

Reinforced concrete tanks are used liquid containing vessels. Such tanks can be ground supported tanks ground storage reservoir and high ground level storage reservoir or elevated water tanks may be referred as elevated storage reservoir. Although most design codes provide guidelines for rectangular and cylindrical tanks, no guidance is provided in EBCS codes for Elevated water tanks. For the analysis and designing the Intze tank along with the EBCS code, ACI code and IS code is used. Therefore, this thesis is study the behavior and design of this type of tanks. In areas with high probability of natural disasters, ability of lifeline systems to resist disaster related damages is one of the most important civil engineering challenges. Elevated water tanks are one of the most important lifeline structures. In this thesis an extensive computational study has been conducted to find out the performance of elevated Intze water tank under wind force. Since these structures have large mass concentrated at the top of slender supporting structure, these structures are especially vulnerable to horizontal forces due to wind. Elevated water tanks are analyzed with different parameters to study the effect of capacity, height of staging, terrain category and wind zone. Findings of the present study shall lead us to better understanding of the behavior of elevated water tank under wind load and safer design of such structure. In doing the design the working stress methods and limit state design is used based on the requirement. In this study membrane analysis is used to find the meridional thrust and hoop stress calculation at various components of the Intze tank.

KEYWORDS: Life line systems, Intze tank, Meridional thrust, Hoop stress, EBCS, ACI, IS codes, Wind load

CHAPTER-I

INTRODUCTION

1.0. Introduction

Water is basic human needs for daily life sufficient water distribution depends on design of a water tank in certain area. Water supply is a life line facility that must remain functional even if disaster occurred. Elevated water tank is a water storage container constructed for the purpose of holding a water supply at a height sufficient to pressurize a water distribution system. In major cities the main supply scheme is augmented by individual supply systems of institutions and industrial estates for which elevated tanks are an integral part. Also at the times of cyclone it was observed that the storage tanks were displaced by few meters and some were overturned due to wind. They were swept away by the wind. Flying debris caused dents on the surfaces when they hit the tanks. So it is important to check the severity of these forces for particular region.

The study of damage histories revealed damage/failure of reinforced concrete elevated water tanks of low to high capacity. Damage of the important lifeline facility like elevated water tanks often results in significant hardships even after the occurrence of the disaster, claiming human casualties and economic loss to build environment. Investigating the effects of wind has been recognized as a necessary step to understand the natural hazards and its risk to the society in the long run. Most water supply systems in developing countries, such as Ethiopia, depend on reinforced cement concrete elevated water tanks. The strength of these tanks against lateral forces, such as those caused by wind, needs special attention.

A water tower also serves as a reservoir to help with water needs during peak usage times. A water tower is an elevated structure supporting a water tank constructed at height sufficient to pressurize a water supply system for the distribution of potable water and to provide emergency storage for fire protection .In some places the term stand pipe is used interchangeably to refer a water tower especially one with tall and narrow proportions. Water towers are able to supply water even during power outages because they rely on hydrostatic pressure produced elevation of water (due to gravity) to push the water into domestic and industrial water distribution systems.

1.1. Need for study of Water Tanks

1) Water tanks are visually simple but structurally difficult
2) Difficult to take the load cases and load combinations
3) Distribution of stress in the structure
4) Distribution of mass
5) Hydro dynamic effects
6) Very critical problem is the slab and beam joints

1.2. Classification of water tanks

1.2.1. In general water tanks can be classified under 3-heads

1) Tanks resting on ground
2) Elevated tanks supported on staging and
3) Underground tanks.

1.2.1.1. Tanks resting on ground

These are used for clear water reservoirs, settling tanks, aeration tanks etc. these tanks directly rest on the ground. The walls of these tanks are subjected to water pressure from inside and the base is subjected to weight of water from inside and soil reaction from underneath the base. The tank may be open at top or roofed

1.2.1.2. Elevated tanks supported on staging

These tanks are supported on staging which may consist of masonry walls, R.C. tower or R.C. column braced together- The walls are subjected to water pressure from inside. The base is subjected to weight of water, weight of walls and weight of roof. The staging has to carry load of entire tank with water and is also subjected to wind loads.

1.2.1.3. Underground tanks

These tanks are built below the ground level such as clarifier's filters in water treatment plants, and septic tanks .The walls of these tanks are subjected to water pressure from inside and earth pressure from outside. The base of the tanks is subjected to water pressure from inside and soil reaction from underneath. Always these are covered at top. These tanks should be designed for loading which gives the worst effect.

The design principles of underground tanks are same as for tanks resting on the ground. But the walls of the underground tanks are subjected to internal water pressure and outside earth pressure. The section of wall is designed for water pressure and earth pressure acting separately as well as acting simultaneously.

1.2.2. Classification of water tanks based on shape

1) Circular tanks
2) Rectangular tanks
3) Spherical tanks
4) Intze tanks and
5) Circular tanks with conical bottoms.

1.2.3. Layout of overhead tanks

Generally the shape and size of elevated concrete tanks for economical design depends upon the functional requirements such as

i) Maximum depth of water
ii) Height of staging
iii) Allowable bearing capacity of foundation strata and type of foundation suitable
iv) Capacity of tank and
v) Other site conditions.

1.2.4. Classification and layout of elevated tanks

Based on the capacities of the tank the possible classification for types of elevated tanks may be as followed as given below for general guidance.

a) For tank up to $50m^3$ capacity may be square or circular in shape and supported on staging on three or four columns.
b) Tank capacity above 50 m^3 and up to $200m^3$ may be square or circular in plan and supported on minimum four columns.
c) For capacity above $200m^3$ and up to 800 $m^3$the tank may be square, rectangular, circular or Intze type tank. The number of columns to be adopted shall be decided based on the column spacing which normally lies between 3.6 and 4.5m
   For circular, Intze or conical tanks a shaft supporting structures may be provided

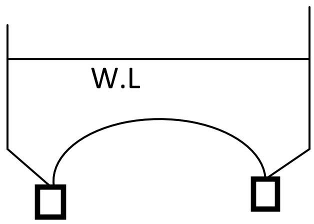
1.3. Intze Tank:

The Intze principle is a name given to two engineering principles both named after the hydraulic engineer Otto Intze. In the one case the Intze principle relates to a type of water tower, in the other a type dam.

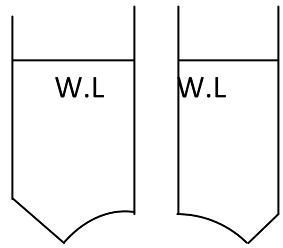
Circular tanks with flat bottom as well as with domical bottom:

In the flat bottom the thickness and reinforcement is found to be heavy. In the domed bottom though the thickness and reinforcement in the dome is normal, the reinforcement in the ring beam is excessive.

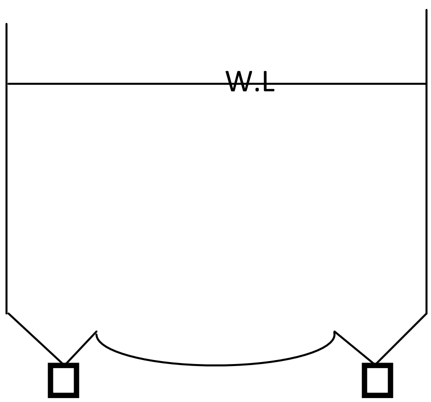
Therefore in the cases of large diameter tanks and economical alternative would be to reduce its diameter at its bottom by conical dome. Such a tank is known as Intze tank and is very commonly used. The main advantage of Intze tank is that the inward radial thrust of the conical bottom balances the outward radial thrust of the spherical bottom. Water tanks designed on the Intze principle



a: Intze 1 Tank



b: Intze 1 tank with inside cylinder



c: Intze 2 Tank

Fig.1.0.Types of Intze tanks.

1.4. Load combinations

Design of liquid retaining structures involves decisions to be made by the engineer based on rules of thumb, judgment, code of practice and previous experience.

1.4.1. Imposed loads

Weight of water may be taken as live load for members directly continuing the same. The weight of water shall be considered as dead load in the design of staging.

1.4.2. Wind load

Wind shall be applied according to EBCS. While analyzing the stresses the combination shall be as follows.

a) Wind load with empty tank and
b) Wind load with tank full.

The worst combination of the stress on account of the above shall be considered while working out the permissible stresses.

The following are the loads and loading conditions which prevail over R.C. water tank

No	Loads	Influence of load on chimney/staging
1.	Dead Load	Static
2.	Live Load	Static+ Dynamic
3.	Wind Load	Static+ Dynamic
4.	Thermal stress	Static
5.	Seismic Load	Static+ Dynamic

Table 1.0. Loading conditions

1.5. Statement of Problem

Lack of expertise in analyzing and designing of elevated water tanks in Ethiopia, The present study is an effort to standardize the analysis and design of this.

1.6. Objective

1.6.1. General objective

- The main objective of this study to identify the dynamic behavior of elevated water tank under wind load.

1.6.2. Specific objectives

- To develop and use the formula for membrane stresses in shells;
- To analyze the stresses in the roof and bottom domes of the tank, the conical section and cylindrical section'
- To analyze the staging from wind load point of view at different heights of staging ; and
- To develop response curves.

1.7. Methodology

- A detailed literature study is done to look into the background of various concepts in previous studies.
- To analyze and design of Intze water tank using EBCS 1995(1, 2, 8), American concrete institute and Euro code (2, 8) 2004.
- Analysis is done by the finite element software SAP 2000 for earthquake
- Produce graphical representation is done wherever required

1.8. Scope

The scope of this research is limited to understanding the behavior of elevated water tanks subjected to dynamic loads such as wind loads and fully study and analyze the membrane theory of shells because the roof and bottom domes of an Intze tank are spherical domes with the shell thickness small compared with the other dimensions and with the radii of curvature.

1.9. Limitations

- This study is not included the soil structure interaction
- Limited to study wind load for different elevations but not for different bearing capacity of soils
- It is assumed that the sloshing wave height is negligible. Sloshing is defined as the periodic motion of force liquid surface in partially filled containers. It is caused by any disturbance of partially filled liquid containers. Sloshing results additional hydrodynamic pressure.

1.10. Literature Review

Up to now many researchers had been contributed on overhead water tank from wind load and earthquake. Therefore the published journals of them are now as reference**s**

Manoranjan shoot and Tandrita bitwise (2007): Study on water tanks in the Kutch region of Gujarat (India) after that area subjected to the earthquake

- They have found in that the tanks in majority of the cases they were designed for the wind load but they were not checked for the earthquake load by assuming that the tanks will resist the earthquake load once they are designed for the wind load.
- They were concluded type of staging is good for resisting the later loads like wind and earthquakes.
- The frame type staging is superior to the shaft type of the staging because the frame types of staging have many flexural members that are provided in the form of columns and braces.

Akshya B.Kamdi and R.V.R.K. Prasad (2012):

- Their contribution in relation to circular water tanks is theory behind the usage of the limit state method and working stress methods and also specified the necessity of the calculation of the crack width.

Pathway: Artificial neural networks were used in predicting the cost of Intze and circular tanks

Mohammed: Application of optimization method to the design of storage tanks was done by

Saxana: Heuristic flexible tolerance method based on the Indian and ACI (building 1969) codes for achieving minimum cost design of an Intze type R.C.C tank presented by.

Jan: A direct search method and the SUMT was used by Jan for finding out minimum cost design of a R.C.C cylindrical water tank based on the British code for water tanks.

Dr. Manoj Hedaoo & Dr. Suchita Hirde [2011]: On the study of seismic performance of the elevated water tank for various seismic zones of India for various heights and capacity of elevated water tanks for different soil conditions

- The effect of height of water tank, earthquake zones and soil conditions on earthquake forces have been presented in this paper with the help of analysis of 240 models of various parameters.

- The study is carried out on RCC circular elevated water tank with C-20 grade of concrete and Fe-415 grade of steel & SMRF are considered for analysis.
- Elevated water tank having 50,000 liters and 100,000 liters capacity with staging height 12 m. 16 m, 20 m, 24 m, 28 m considering 4 m height of each panels are considered for the study.

Author has given following conclusions from his analysis

- Seismic forces are directly proportional to the Seismic Zones,
- Seismic forces are inversely proportional to the height of supporting system,
- Seismic forces are directly proportional to the capacity of water tank, and
- Seismic forces are higher in soft soil than medium soil, higher in medium soil than hard soil. Earthquake forces for soft soil is about 40-41% greater than that of hard soil for all earthquake zones and tank full and tank empty condition.

Now a days the population growth of urban area of Ethiopia increases. Because of this the demand of sufficient and clean water supply at peak hour and during power shortage time is very crucial.

- To minimize this problem high raised, large and Intze shape of overhead tanks are a better choice for pressurized a water distribution system.

CHAPTER-2

MATERIALS USED AND THEIR DESIGN REQUIREMENTS

2.1. CONCRETE

Reinforced concrete structures are of different because the design of liquid retaining structure is different from an ordinary R.C. Structure as it is required that the concrete should not crack. In order to make the liquid retaining structure efficient working it should be of high strength and quality and should be leak proof. The design of the concrete mix should be done in such a manner that the resultant concrete is sufficiently impervious. Also at compaction level efficient compaction preferably by vibration is essential. Therefore the thoroughly compacted concrete permeability is dependent upon water cement ratio. Water cement ratio and permeability are directly proportional that is Increase in water cement ratio increases permeability, while concrete with low water cement ratio is difficult to compact. Thus water cement ration is to chosen in such a manner that the compacted concrete have sufficient permeability and good workability. The maximum free water cement ratio for liquid retaining structures shall be 0.45 for reinforced concrete and 0.50 for plain concrete. Lower water cement ratio may be achieved by us in suitable admixtures like plasticizers or super plasticizers. The Amount of such plasticizer shall not be more than 2% by mass of cementations material (I.S: 10262-2007), i.e., sum of mass of cement and additives.

Not only had the above said the other causes are also there for leakage in concrete. They are defects such as segregation and honey combing. Along these joint should be given proper care. Because all joints should be made water-tight as these are potential sources of leakage. Over these certain measures will help to make the water retaining structures to be efficient. Use of small size reinforcement bars placed properly, leads to closer cracks with smaller width. The risk of cracking because of temperature and shrinkage effects may be minimized by limiting the changes in moisture content and temperature to which the structure as a whole is subjected. To control the shrinkage and thermal movements' provision of joints deserves extra special attention in case of liquid retaining structures.

Generally concrete mix weaker than C-30 is not used. Considering the concept of durability of water retaining structures this is the minimum grade of concrete. In order to get high quality and impervious concrete, the proportion of cement, fine and coarse aggregate to is determined carefully and water cement ratio is adjusted accordingly. Finally depending up on the exposure conditions of the structure, the grade of concrete is decided .Minimum cement content excluding the additives like fly ash and granulated slag shall not be more than 400 kg/m3 to safeguard against cracking due to drying shrinkage.

2.2. STEEL

Since concrete and steel are assumed to act together, therefore it is required to check the stresses in the steel and concrete. From concrete point of view it has to be checked to avoid cracks in concrete whether the tensile stress in concrete is within limits or not .On the other hand the tensile stress in steel will be limited by the requirement that the permissible tensile stresses in concrete is not exceeded. For calculation of strength the permissible stresses in steel reinforcement is as follows.

(a) Permissible tensile stresses in member in direct tens = 1500 Kg/Cm2
(b) Tensile stress in member in bending

On liquid retaining face of member = 1500 Kg/Cm2

On faces away from liquid for members less than 225 mm thick = 1500.Kg/Cm2

(c) On faces away from liquid for members 225 mm. thick or more = 1900 Kg/Cm2

2.3. Minimum Reinforcement

Minimum reinforcement is based on the surface zones of the member. When the thickness of the member is up to 500 mm, i.e.,$\leq 500\ mm$, assume each surface zone to be of thickness equal to $\frac{D}{2}$. When the thickness of the member exceeds 500mm assume each surface zone of 250mm thickness and the internal concrete shall be ignored for the purpose of the minimum reinforcement calculations. Minimum reinforcement in walls, floors and roof in both the perpendicular directions shall not be less than 0.35% of the surface zone cross section for HYSD bars and not less than 0.64% for mild steel bars. If the length the member is less than 15 m these reinforcement can be reduced to 0.24% for HYSD bars and 0.4% for mild steel bars. If the thickness of the wall is less than 200mm, the calculated reinforcement may all be placed on one face.

For slabs up to 500mm depths all the steel calculated shall be provided equally on both sides. This steel is required against temperature strain across the depth of the slab. Temperature strains are much smaller at the center of depth. Therefore for the depth more than 500 mm the central part is ignored. Thus for slab depth more than 500mm, the minimum reinforcement remains constant with depth at both faces. The spacing of the bars shall not be greater than 300mm or thickness of the section. And also permissible stresses in steel in working stress method are reduced to decrease the tendency of cracking.

The area of the reinforcement shall be such that when crack forms the reinforcement shall be able to absorb the total force. If the steel ratio is lower the ultimate concrete strength will be more than strength of steel. If crack forms the steel will yield and will not be able to properly restrain the crack width. On the other hand if steel ratio is more the steel will not yield and resist all tensile force leading to limited crack width.

In the circular part of the tank hoop steel is main steel (horizontal) and is circular in shape. And the other one vertical is the distribution steel. In order to reduce the labor of fixing the reinforcement horizontal steel will be placed in outer layers. Therefore the horizontal steel should be properly curved in a hooping machine and not bent series of kinks. And also one bar cannot be used continuous for perimeter of the wall. Therefore lapping of bars is necessary.

For an ordinary slab or beam generally lapping is done at the place where stresses are reduced to 50%. In circular portion of the tank walls, at all the sections of the horizontal bar the stress is the same. Therefore it is preferable to carefully plan the splicing of reinforcement to see that splices are very well staggered. In the case of lapping the lap length should be equal to"$2L_d$". As usual all the bars should not be staggered at one section and should be staggered. It should be taken care that at any vertical section not more than one bar in three bars should be lapped. At any vertical section axial tension is shared by concrete and steel, however at the vertical joint only steel resists tension and therefore steel is provided to carry all the tension. The bars should not be lapped at or near the vertical joint.

2.4. Concrete cover

For liquid retaining structures, the exposure is considered to be “severe”. The minimum concrete cover to the reinforcement shall be 45 mm.

CHAPTER-3

ANALYSIS and DESIGN INTZE TANK

3.0. Introduction

Working stress method and Limit state method

The working stress method of design was evolved around 1900. Although this method have been performing its function satisfactorily since a long time but this method of design results in a larger percentage of steel and uneconomic sections. However the working stress method is the only method available to check the R.C sections against failing in service stresses and serviceability states of deflection and cracking. But the modern methods i.e., limit state design which is based on a semi probabilistic approach. Although this method provides much economical and safe sections but it has not been formulated yet to suit the design of structures like storage tanks. Limit state method allows higher strain in steel as well as in concrete which creates the problem of cracking in R.C. structures. Hence working stress method is still used in the design of tanks.

The components of the water tank (container) are designed by using working stress method. Other related elements like columns, ties, stair (If provided of an R.C) designed according to the limit state method.

3.1. Analysis and Design of elevated water tanks

This study has emphases merely on elevated water tank. Design of liquid retaining structures has to be based on the avoidance of cracking in concrete regard to its tensile strength. It has to be ensured that no cracks should be formed on the water face. The design of such tanks is done in two ways.

1) Membrane analysis
2) Analysis taking into account of continuity effect at joints.

In the membrane analysis it was assumed that each member is independent of the other and therefore subjected to direct stresses only and no bending moment is introduced. However due to continuity of joints between the members joint reactions are introduced due to which secondary stresses in the form of edge moment and hoop stresses are introduced in the members. Stresses due to continuity can be obtained by applying the principle of consistent deformations. At each joint the horizontal deformation and angular displacement between the shells should be consistent

The analysis of Intze tank is therefore done in two stages

1) Membrane analysis in which membrane stresses in each member are calculated and the members are designed
2) Analysis of effects due to continuity in which the deformations due to membrane stresses are first calculated and equations of consistent deformations are formulated to know the secondary stresses.

The final stresses are then found by adding the stresses due to the above two cases.

The main advantage of such a tank is that the outward thrust from the top of the conical part is resisted by the ring beam B3 (showed in below figure), while the difference between inward thrust from the bottom of the conical dome and the outward thrust from the bottom dome are resisted by ring beam B2 (shown in below figure).

The Proportions of the conical dome and the bottom dome are so arranged that that the outward thrust from bottom dome balances the inward thrust due to the conical dome.

The below figure suitable proportions for Intze tank with internal diameter "D". The volume of water stored in the tank with those proportions is $0.585D^3$

In general the volume of water stored is given by

$$V = \frac{\pi}{4}D^2h + \frac{\pi h_o}{12}\left(D^2 + {D_o}^2 + DD_o\right) - \frac{\pi {h_2}^2}{3}(3R_2 - h_2)$$

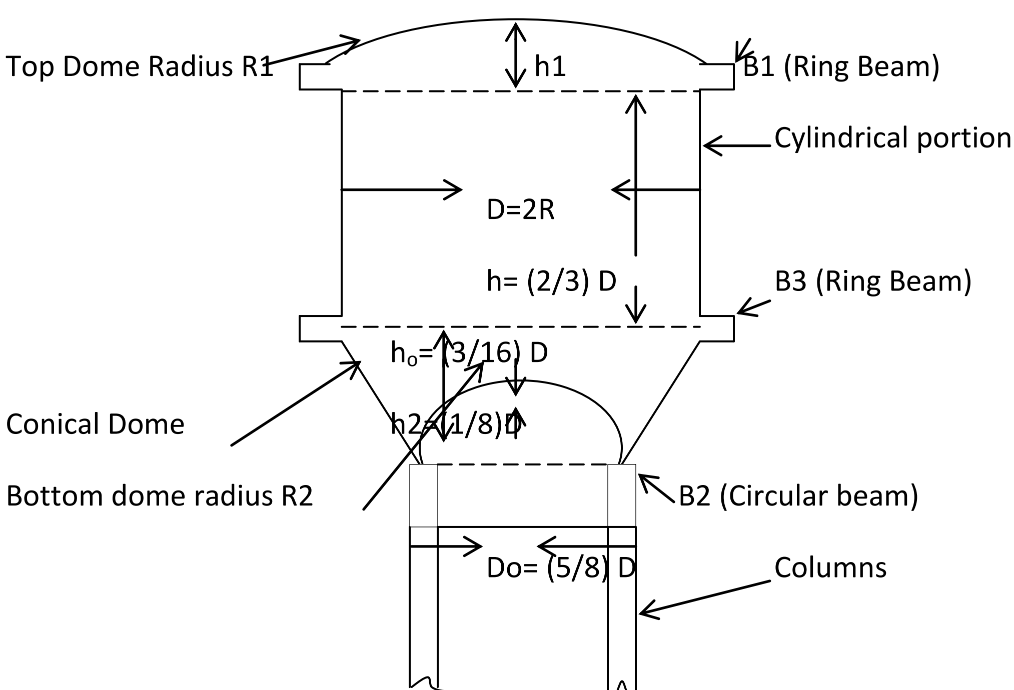


Fig.3.0. Components of Intze tank

From economical considerations the inclination of the conical dome should be $50^0\ to\ 55^0$ with the horizontal.

3.1.1. Membrane analysis

In the membrane analysis the members are assumed to act independent of the others. The members are therefore subjected to only direct stresses and no B.M is introduced.

3.1.2. Top dome and Top Ring beam B1

A dome may be defined as a thin shell generated by the revolution of a regular curve about one of its axes. The shape of the dome depends upon the type of the curve and the direction of the axis of revolution. When the segment of curve revolves about its vertical diameter a spherical dome is obtained

Similarly conical dome is obtained by the revolution of the right angled triangle about its vertical axis. While the elliptical dome is obtained by the revolution of a right angled triangle about one of its axes.

However out of these spherical domes are more commonly used. In case of a spherical dome the vertical section through the axis of revolution in any direction is an arc of circle

Domes are used in variety of structures such as

1) Roof of circular areas
2) Circular tanks
3) Hangers
4) Exhibition halls, auditoriums and planetariums and
5) Bottoms of tanks , bins and bunkers

Nature of stresses in spherical domes:

1) A spherical dome may be imagined to consist of a number of horizontal rings placed one over the other
2) The diameter of successive rings increases in the downward direction and the equilibrium is maintained independently of the rings above it.
3) The circle of each ring is called latitude
4) The circle drawn through two diametrically opposite points on a horizontal diameter and the crown is known as a meridian circle. All meridian circles converge at the crown (or top most point) of the spherical dome.

The below (b) shows the vertical section of the spherical dome. The successive horizontal rings subtend equal angle at the center of the sphere. The joint between successive horizontal rings is radial. Every horizontal ring supports the load of the ring above it and transmits it to the one below it. The reaction between the rings is tangential to the curved surface giving rise to compression along the medians. The compressive stress is called meridonial thrust or meridonial compression.

The below C shows the plan of a horizontal ring which may be imagined to consist of a number of voussoirs. The joints between adjacent voussoirs of the ring are radial. The tendency of separation of any voussoir will be prevented because of its wedge shape and therefore hoop compression will be caused in each ring.

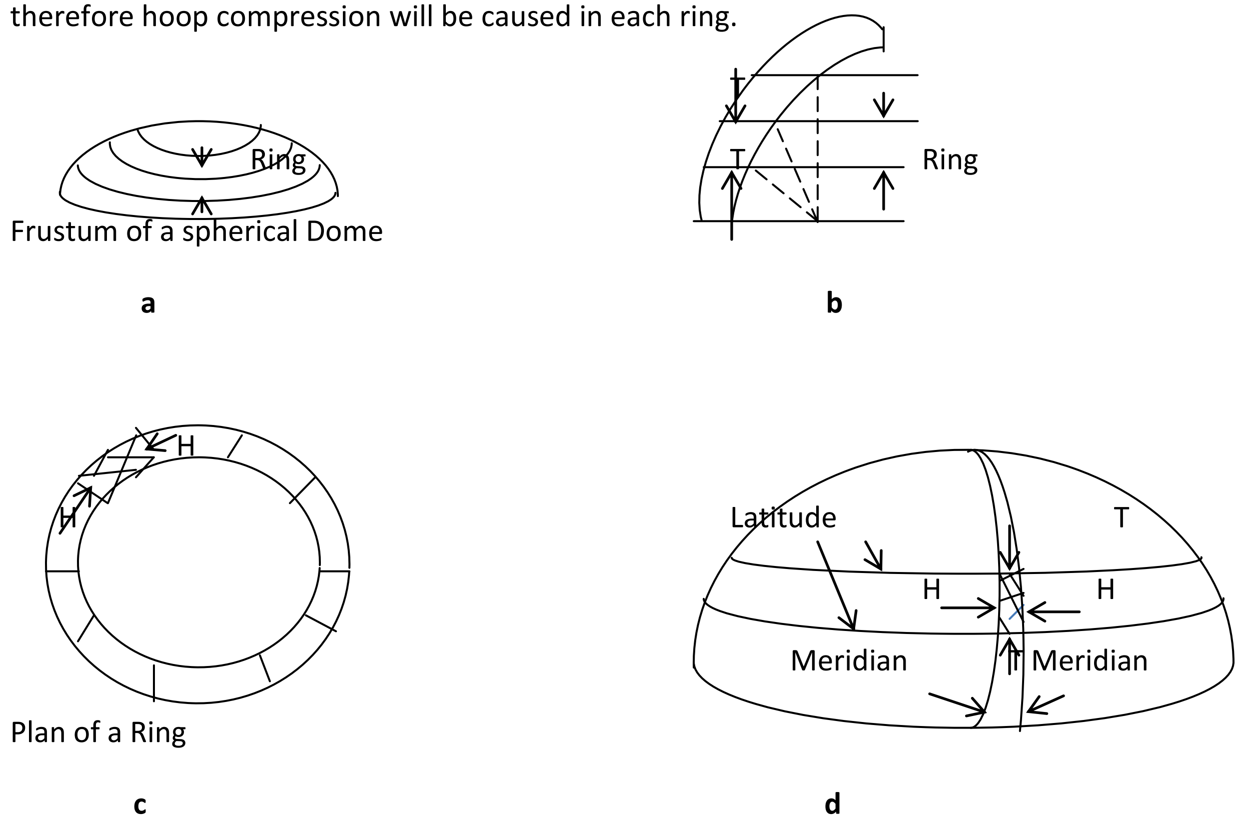


Fig.3.1. Sectional location of various forces

To summarize therefore two types of stresses are induced in a dome.

1) Meridonial thrust(T) along the direction of meridian
2) Hoop stress along the latitudes.

Analysis of spherical domes:

Let us now analyze stresses developed in a spherical dome of uniform thickness for uniformly distributed load.

w- Uniformly distributed load inclusive of its own weight per unit area r- Radius of the dome

t- Thickness of dome shell

T- Intensity of meridonial thrust

H- Intensity of hoop stress.

Meridionial Thrust:

The fig (4) shows the section through the vertical axis of revolution of a thin spherical dome

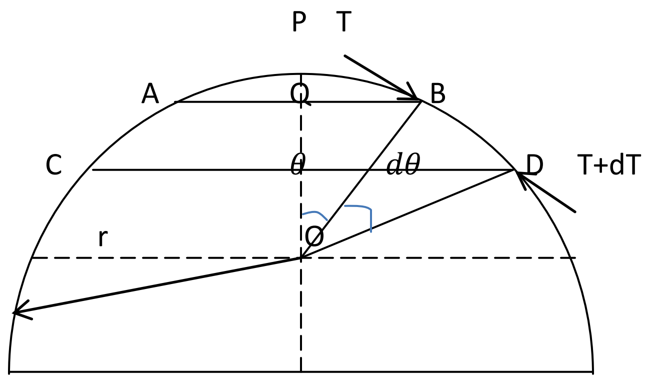


Fig.3.2. Variation of meridonial thrust over a sectional dome

Let us consider the equilibrium of a ring ABDC, between the two horizontal planes AB and CD. The extremity of the horizontal plane AB makes an angle “θ" with the vertical at the center. While the extremity of the horizontal plane CD makes an angle"$\theta + d\theta$". The ring thus subtends an angle "$d\theta$" at the centre.

The following are the forces acting on the unit length of the ring

1) The meridonial thrust “T” per unit length of the circle of latitude “AB” acting tangentially at “B”(or at right angles to the radial line “OB”)
2) The reaction or thrust “T+dT” per unit length of circle on latitude “CD” acting tangentially at “D”
3) The weight “δw" of the ring itself acting vertically down

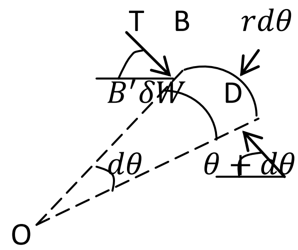


Fig.3.3. Forces acting on the unit length of the ring

It should be noted that the reaction "T+dT" will be greater than the thrust "T" due to the effect of the weight of the ring and due to change in the inclination from " θ" of "$\theta + d\theta$". Of the radial lines.

The meridonial thrust "T" is caused due to the weight of dome shell APB above the rotational plane "AB"

Surface area of dome shell APB =$(2\pi r)(PQ)$

But $PQ = OP - OQ$

$$= r - rcos\theta = r(1 - cos\theta)$$

Weight of dome shell above $AB = 2\pi r(PQ)(w)$

$$= 2\pi r(r(1 - cos\theta)(w)$$

$$= 2\pi r^2 w(1 - cos\theta)$$

Since the sum of vertical components of thrust "T" acting along the circumference of the circle of latitude must be equal to the total weight of the dome shell "APB" we have

$$T(2\pi)(QB)sin\theta = 2\pi r^2 w(1 - cos\theta)$$

$$T(2\pi)\ rsin\theta(sin\theta) = 2\pi r^2 w(1 - cos\theta)$$

$$T = \frac{wr(1 - cos\theta)}{sin^2\theta} = \frac{wr}{1 + cos\theta}$$

3.1.3. Hoop stress

We have seen the meridonial thrust "T" increases to "T+dT" at the bottom of the ring. This difference in the meridonial thrust "T" and "T+dT" acting at "θ" and "θ+dθ".respectively to the horizontal causes hoop stress.

Let “H” be the hoop force per unit length of surface measured on a great circle arc:

Breadth of ring = $rd\theta$

Hoop force = $Hrd\theta$

The horizontal components of “T” is "$Tcos\theta$" and this horizontal component cause hoop tension tending to increase the diameter of the ring .While horizontal component of “T+dT” will (T+dT) cos (θ+ dθ) and this horizontal component cause hoop compression

Now magnitude of hoop tension = $Tcos\theta(Radius\ of\ ring\ AB)$

$$= Tcos\theta(rsin\theta) = Tr\ sin\theta\ cos\theta - - - - - - - - (1)$$

Magnitude of hoop compression=$(T + dT)\cos(\theta + d\theta)\ radius\ of\ ring\ CD$

$$=(T + dT)\cos(\theta + d\theta)\, r\sin(\theta + d\theta) - - - - - - -(2)$$

The difference between (1) & (2) specify the resultant stress

If 1 > 2---- hoop tensile

1 < 2------ hoop compression

Hence the limiting case when "$d\theta$" is extremely small

$$Hrd\theta = d(T\ cos\theta\ r\ sin\theta)$$

But $T = \frac{wr(1-cos\theta)}{sin^2\theta}$

$$H = \frac{1}{r}\frac{d}{d\theta}\left[r\ sin\theta\ cos\theta\ \frac{wr(1-cos\theta)}{sin^2\theta}\right]$$

$$= wr\frac{d}{d\theta}\left[\frac{1-cos\theta}{sin^2\theta}\ sin\theta\ cos\theta\right]$$

$$= wr\frac{d}{d\theta}\left[\frac{cos\theta}{sin\theta} - \frac{cos^2\theta}{sin\theta}\right]$$

$$H = wr\left[\frac{-sin^2\theta - cos^2\theta}{sin^2\theta} - \frac{-2sin^2\theta\ cos\theta - cos^3\theta}{sin^2\theta}\right]$$

$$Therefore\ H = \frac{wr(cos^2\theta + cos\theta - 1)}{1 + cos\theta}$$

The above expression gives the hoop stress in any horizontal ring the extremity of which subtends an angle "θ" with the vertical at the center. If the value of "H" obtained from above equation is positive hoop force will be compressive otherwise it will be tensile.

At the crown$\theta = 0$, hence equation becomes $H = \frac{wr}{2}$

Intensity of hoop stress at crown is $\frac{H}{t} = \frac{wr}{2t}$ (compressive), This is the maximum value of hoop stress. The hoop stress goes on decreasing as "θ"increases till "H" becomes zero. After that "H" becomes tensile.

To find the position of the plane where hoop stress becomes zero we have

$$H = 0 = \frac{wr(cos^2\theta + cos\theta - 1)}{1 + cos\theta} \; or \qquad cos^2\theta + cos\theta - 1 = 0$$

$$or\; cos\theta = 0.618\; for\; which\; \theta = 51^0 49' 38''$$

Hence round the circle of latitude at which the angle $\theta = 51^0 49' 38''$ hoop stress is zero. For all portion of dome about this angle hoop compression will be developed while for the portion below this plane hoop tension will be developed which will go on increasing further towards the base of the dome.

3.2. Design of R.C Domes

The requirements of thickness of dome and reinforcement from the point view of induced stresses are usually very small. However a minimum thickness of 7.5 cm is provided to protect the steel. Similarly minimum steel provided is 0.15% of the section area in each direction meridonally as well as along the latitudes. This reinforcement will be in addition to the hoop tensile stresses. The steel reinforcement is provided in the middle of the thickness of the dome shell. Near the edges some hogging bending moment may be developed and hence meridonial steel should be placed near the top surface.

For Cover slab: It may be flat or in domed shape. For small plan area, the cover slab may be flat, however for large area, domes are economical

3.2.1. Placement of main reinforcement in dome

As stated earlier a minimum reinforcement of 0.15% of area is provided both in the direction of latitude as well as of the meridians. If the reinforcement along the meridians is continued up to crown there will be congestion of steel there. Hence from practical considerations the meridonial reinforcement is stopped at any latitude circle near crown and a separate mesh is provided. No separate reinforcement along latitude is provided in this area at the crown

3.3. Provision of ring beam

If the dome is not hemispherical the meridonial thrust at the supporting circle of latitude (i.e., at the base) will not be vertical. The inclined meridonial thrust at the support will have horizontal component which will cause the supporting walls to burst outwards causing its failure. In order to bear this horizontal component of meridonial thrust a ring beam is provided at the base of the dome.

The reinforcement provided in the ring beam takes this hoop tension and transfer only vertical reaction to the supporting walls. The tensile stress on the equivalent area of concrete on the ring beam section should not exceed 12 N/mm^2

3.4. Provision of openings

Openings may be provided in the dome as required from other functional or architectural requirements. However sufficient trimming reinforcement should be provided all round the openings as showed below. The meridonial and hoop reinforcement reaching the opening should be well anchored to the trimming reinforcement.

If there is an opening at the crown of the dome and if there is any concentrated load of lantern etc. acting there a ring beam should be provided at the periphery of the opening

The design is carried out as per relevant analysis procedures combined with Indian standard codes of practices.

The water tank dome is designed by working stress method and the supporting columns and braces by limit state method. The analysis account for all forces inside the dome arising out of water retained and live loads including the external environmental forces of wind in addition to ubiquitous dead loads.

The foundation forces at the level of safe bearing capacity are also evaluated and then foundation design can be done.

3.5. The cylindrical portion of tank

Let the diameter of the tank “D” and the height of the cylindrical portion “H”. The walls are assumed to be free at top and bottom. Due to this tank walls will be subjected to hoop tension only without any bending moment, maximum hoop tension will occur at base, it’s magnitude being equal to $\frac{whD}{2}$ per unit height. The tank walls are adequately reinforced with horizontal rings provided at both faces .In addition to this vertical reinforcement is provided on both the faces in the form of distribution reinforcement.

3.6. Ring beam B3 at the junction of cylindrical wall and conical dome

The vertical load at the junction of wall with conical dome is transferred to ring beam B3 by meridional thrust in the conical dome. The horizontal component of this thrust causes hoop tension at the junction. The ring beam is provided to take up this hoop tension

W-Load transmitted through tank wall at the top of conical dome per unit length

$\emptyset_o$ –Inclination of conical dome with vertical

T- Meridional thrust in conical dome at the junction

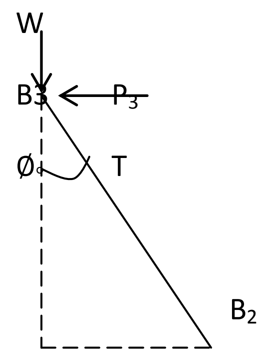


Fig.3.4. Loads at the junction of ring beam B3

3.7 .Bottom dome

Domes are economical than flat roofs for large spans. The bottom slab is divided into conical dome and spherical dome in such a way that the inward thrust due to conical dome on bottom most ring beam gets balanced by the outward thrust of the spherical dome. The inclination of the spherical dome is usually $45^0$to 55^0 with the vertical so as to obtain the net thrust as hoop compression and not the hoop tension. The conical dome is used in order to reduce the diameter of the spherical dome. And the diameter of the spherical dome is usually 65% to 75% of the diameter of the tank. The top domes shall be designed for live load of1.5 KN/m2

Bottom dome develops compressive stresses both meridionally as well as along hoops due to weight of water supported by it and also due to its own weight

Let H_o –be the total depth of water above the edges of the dome

The weight of water above the surface of the dome is given by

$$W_o = \left(\frac{\pi {D_o}^2}{4} H_o\right) - \frac{\pi {h_2}^2}{3}(3R_2 - h_2)(w)$$

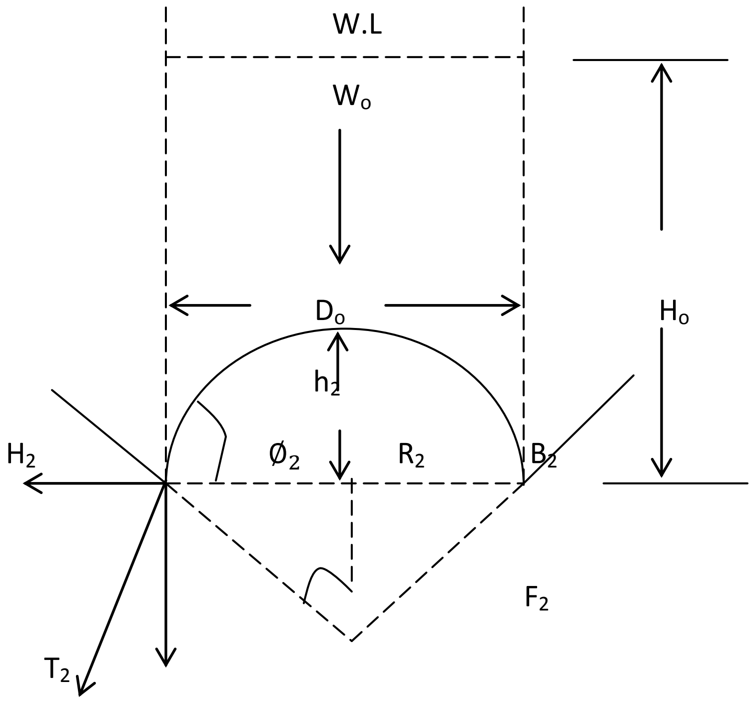


Fig.3.5. Loads on the bottom dome

Where R_2 —is the radius and

h_2 —is the rise of the bottom dome

Total surface area of dome $= 2\pi R_2 h_2$

Self-weight of dome $= 2\pi R_2 h_2 t_2(\gamma_c)$

Where t_2 is the thickness of bottom dome

Total load $= W_T = W_o + 2\pi R_2 h_2 t_2(\gamma_c)$

Meridional thrust $T_2 = \frac{W_T}{\pi D_o sin\varphi_2}$

Intensity of load $P_2 = \frac{W_T}{2\pi R_2 h_2}$

Maximum hoop stress at center $= \frac{P_2 R_2}{2t_2}$

Knowing the meridional thrust and hoop stress the dome can be designed.

3.8. Bottom ring beam B2

The ring beam receives an inward inclined thrust T_o from the conical dome and an outward thrust T_2 from the bottom dome. The horizontal components of both of these oppose each other.

Net horizontal force "P" is given by.

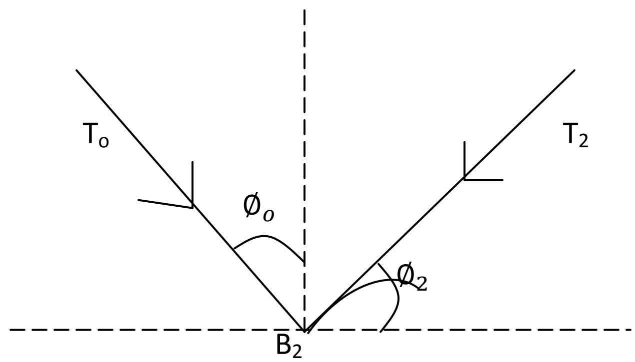


Fig.3.6. Forces at the ring beam B2

$$P = T_o sin\emptyset_o - T_2 cos\emptyset_2$$

If $T_o sin\emptyset_o > T_2 cos\emptyset_2$ the beam will be subjected to hoop compression

If however $T_2 cos\emptyset_2 > T_o sin\emptyset_o$ it will be subjected to hoop tension

Therefore the dimensions of the tank should be so adjusted that either "P" is zero or "P" is compressive.

The hoop force is given by $P_H = P\frac{D_o}{2}$

If b_2 is the width and d_2 is the depth of the ring beam the stresses is given by

$$P_H = P\frac{D_o}{2}\frac{1}{b_2 d_2}$$

The vertical load per unit length is given by

$P_V = T_o cos\emptyset_o + T_2 Sin\emptyset_2$ Per unit length.

The circular ring beam can now be designed for the above superimposed load.

CHAPTER-4

STAGING

4.0. Introduction

Height of staging is the difference between the lowest supply level of tank and the average ground level at the tank site.

For small capacity say 40000 to 50000 liters tanks square in plan are economical. For large capacity water tanks circular tanks prove economical. Among large capacity circular tanks, Intze tanks are economical.

4.1. Design of elevated tanks

Structural design of an elevated water tank consists of:

i) Design of tank (container)

ii) Design of staging

iii) Design of foundation.

4.1.1. Design of tank

Design of tank: Design of water tank (container) consists of designing of elements like cover slab, side walls, base slabs and beams.

4.1.2. Design of staging

The staging for elevated tanks is designed for the following loading conditions.

DL + LL + water load.

DL (Tank empty) +wind load.

DL + LL + water load+ WL

Analysis of Wind load is carried by either the exact methods or approximate methods like portal frame method or cantilever method.

The columns are tied by tie beams for the following reasons:

1) In order to reduce the effective length of the columns.
2) In order to reduce the moments and shears caused due to horizontal loads.
3) Integral action is secured by tying all the columns.

Tie beams are to be designed for axial thrust, shears and moments. The axial thrust from gravity loads shall be considered as 3% of the axial gravity loads in columns. This may be of tension or compression. Forces because of the wind load are obtained from the analysis. As the wind may reverse their directions the forces in the ties will be reversed. Therefore the reinforcement in ties shall consist of top and bottom reinforcement equally distributed.

Staging- Columns and bracings:

Design of staging consists of design of columns and design of bracings. The design will be carried out by using limit state method.

4.2. Design of columns

Gravity loads: Gravity loads on column consist of dead loads and water load. Thus loads on column are determined for tank empty and tank full conditions.

4.2.1. Wind loads

The wind loads produce tension on windward columns, compression in leeward columns and no axial force in columns on the line of neutral axis.

4.2.2. Axial forces in columns

Wind forces on windward side are calculated on container on columns and on bracings. To determine the axial forces in columns, determine the sum of moments of all these forces about the neutral axis at the bottom of the columns.

The staging acts as vertical cantilever supported at the base and subjected to the horizontal wind forces.

4.3. Design of tank supporting towers

The designer before taking up the design should first decide the most suitable type of staging of tanks and correct estimation of loads including statically equilibrium of structure particularly in regard to overturning of overhanging members shall be made. The design is to be based on the worst possible combination of load, moments and shears because of vertical loads and horizontal loads acting in any direction when the tank is empty as well as full.

In order to obtain the desired head of water, water tanks are generally elevated above the ground. This is accomplished either by supporting the tank on masonry walls provided up to the desired height or by supporting it on a number of columns suitably braced at various heights. In the latter case the columns are subjected to

1) Dead load of tank, water and other connected structures.
2) Wind loads or seismic forces.

Generally size of the all the columns are of equal dimensions and are placed symmetrically .Therefore the dead load may be assumed to be equally distributed amongst the columns. The force due to wind and other horizontal loads will depend upon the arrangement of columns and their support conditions.

The loads from the water tank are transferred to the staging through the ring beam, and this ring beam is supported by the columns (staging). Usually 4 to 12 columns are used in the staging they are spaced at equidistance, therefore they share the gravity load equally. For 3 column staging the stresses are very high therefore it is rarely built. The columns may be designed as vertical or with some batter, particularly for tall water tanks the staging is having some batter. This may be in the range of 1:12 to 1.25:12. Wind force may act in any direction but the wind force in the direction parallel to the diagonal works out to be critical. In the columns the compression force develops because of tank loads and overturning moments caused by wind.

We shall consider several cases and analyze them by approximate methods only

4.3.1. Case 1

Rows of columns (Two equal columns) with rigid top and fixed at the footings.

Below fig (9) showing that a tank supported on two equal columns .Let ”P”   be the total wind load on the tank surface. The columns are fixed at the base and are rigidly connected to the tank.

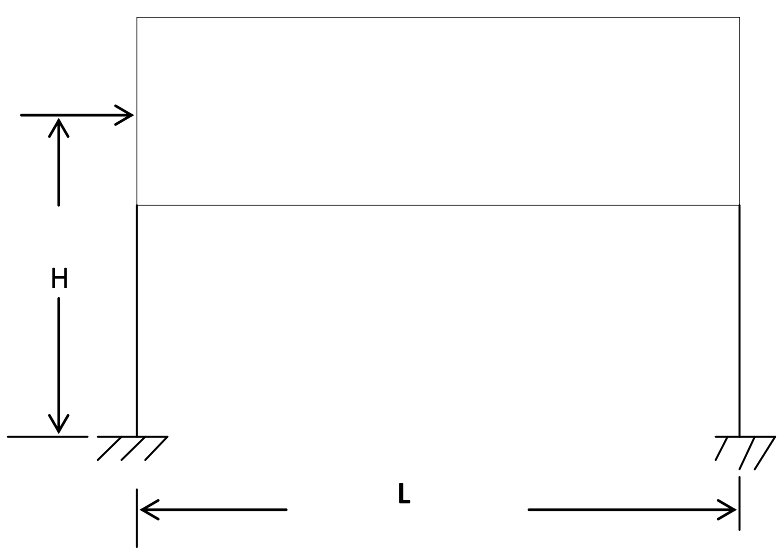


Fig.4.0 Tank supported on two columns

Below figure shows the deflected shape.

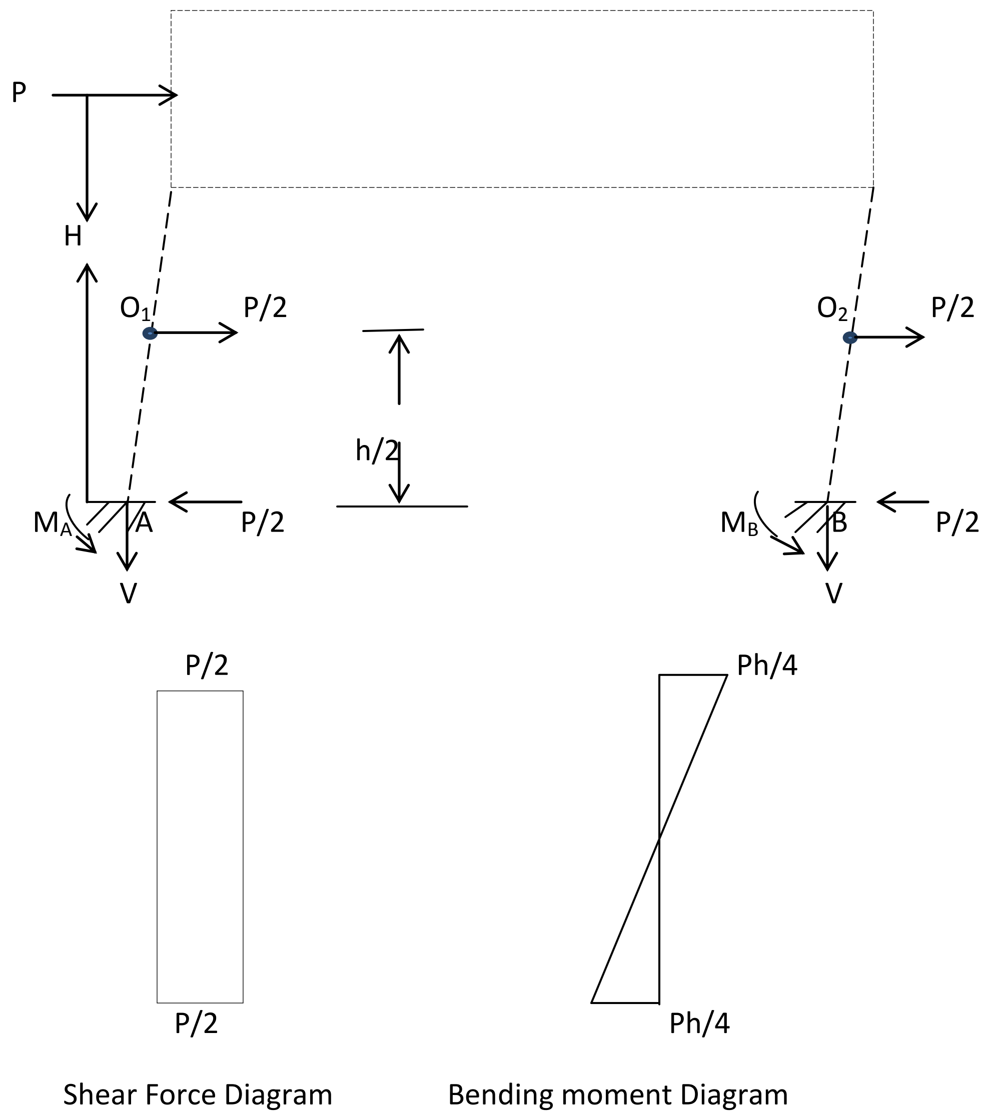


Fig.4.1. Deflected shape of the tank supported on the two columns

The analysis is based on the assumption that the point of contra flexure (o_1 and o_2) occurs at the mid height of each column. At the point of contra flexure there is no bending moment and the column is subjected to only horizontal shear (Q) and axial force (V)

In general there are three effects of wind and other horizontal forces

1) Bending moment –M
2) Horizontal shear –Q
3) Axial Force "V"

At the base of each column the bending moment isM_A. Horizontal shear is $\frac{P}{2}$ and axial force is "V" which is tensile in column "A" and compression in column "B"

Taking moments of external forces about "B" we get,

$$P(H) - V(L) - M_A - M_B = 0 - - - - - - (1)$$

However considering the equilibrium of O, A

$$M_A = \frac{P}{2}\frac{h}{2} = \frac{Ph}{4}$$

Similarly $M_B = \frac{Ph}{4}$

Hence from equation (1)

$$V = \frac{P(H) - \frac{Ph}{2}}{L} = \frac{P}{L}(H - \frac{h}{2})$$

If there are "n" columns in each row, we have $M_A = M_B = \frac{Ph}{4n}$

$V = \frac{P}{nL}\left(H - \frac{h}{2}\right)$ and the shear in each column $Q = \frac{P}{2n}$

The total stress in each column is that due to

1) Dead load of the structure and the contents
2) Axial force $\pm V$
3) Flexural stress due to "M" and
4) Shear stress due to shear "Q" which is considered to be negligibly small

(In the above case the wind load on the column faces has not been considered.)

4.3.2 Case2

Two rows of columns with horizontal braces. The below fig-11 shows a tank supported on two columns (or two rows of columns) subjected to a horizontal wind load "P" on the exposed tank surface. Here also again the wind load on exposed column faces has been neglected for simplicity.

Two rows of columns have been connected with horizontal braces. It is assumed that the braces are so stiff that the columns are constrained to maintain their axis vertical at their junctions with braces.

It is also assumed that columns develop points of contra flexure there will be only horizontal shear (=P/2, in the present case) and axial force, the bending moment being zero.

The bending moment at the junction of the column with the brace such as point "C" will be given by.

$$M_{CE} = \frac{P}{2}\frac{h_2}{2} = \frac{Ph_2}{2} \textit{ above the brace and}$$

$$M_{CA} = \frac{P}{2}\frac{h_3}{2} = \frac{Ph_3}{4} \textit{ below the brace}.$$

Moment in the brace will be the sum of the two

$$M_{CD} = \frac{Ph_2}{4} + \frac{Ph_3}{4} = V_m \frac{L}{2}$$

$V_m = \frac{P(h_2+h_3)}{2L}$

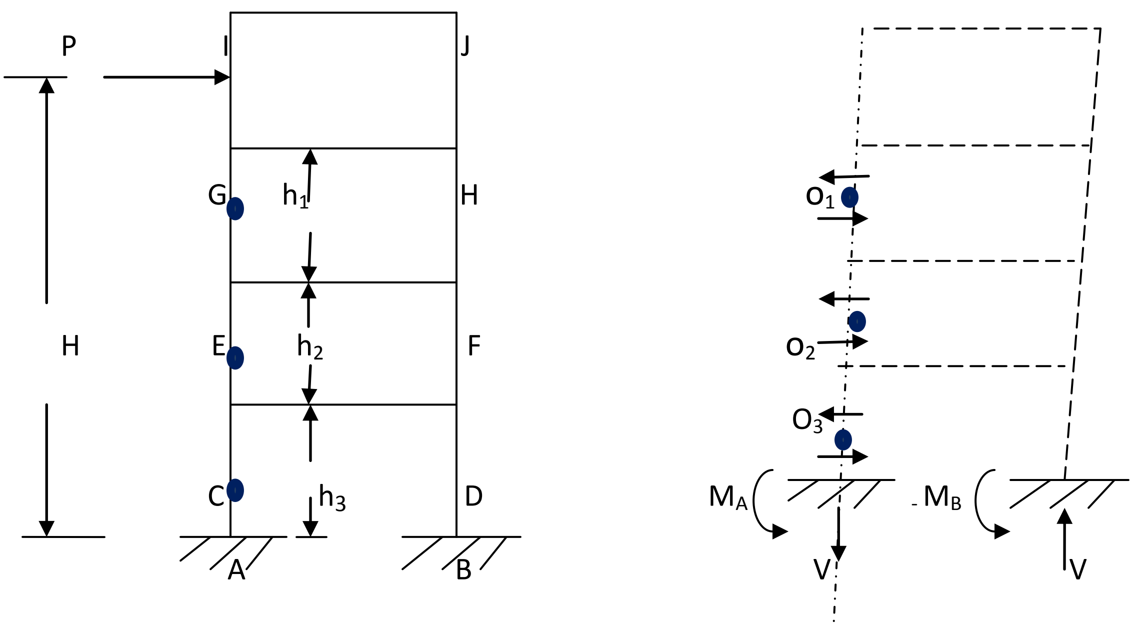


Fig.4.2. Staging subjected to wind

The moments at A and B evidently be

$$M_A = M_B = \frac{P}{2}\frac{h_3}{2} = \frac{Ph_3}{4}$$

To find axial force "V" takes moments about "B"

$$P(H) - V(L) - M_A - M_B = 0$$

$$V = \frac{P(H) - \frac{2Ph_3}{4}}{L} = \frac{P}{L}\left(H - \frac{h_3}{2}\right)$$

If there are “n” columns in each row the above expressions are modified as follows.

$$M_A = M_B = \frac{Ph_3}{4n}$$

$$V = \frac{P}{nL}\left(H - \frac{h_3}{2}\right)$$

4.3.3. Case 3

Frame work with three or more rows of columns

Below fig (12) shows the frame work with three rows of columns having “n” columns in each row and stiffened with braces.

Let “P” be the total load due to wind on the exposed surface of the tank. Since the interior columns “C” is braced on both sides and is held more stiffly than the exterior ones they are assumed to take double horizontal shear than the exterior ones. Thus the horizontal shear at the points contra flexure in each external column will be $\frac{P}{4n}$ while that in each middle column will be $\frac{P}{2n}$.

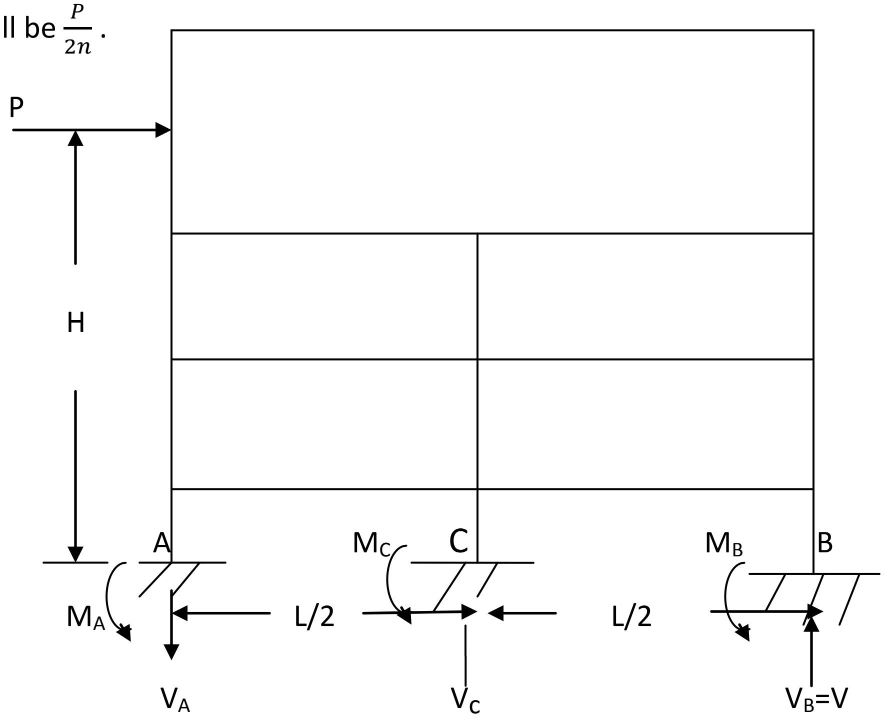


Fig.4.3. Wind force on three or more columns staging

If h_o is the height of the lower panel

$M_A = M_B = \frac{P}{4n}\frac{h_o}{2} = \frac{Ph_o}{8n}$and

$$M_C = \frac{P}{2n}\frac{h_o}{2} = \frac{Ph_o}{4n}$$

The whole frame work will rotate about the horizontal axis passing through "C". Hence the vertical (axial) force in "A' and "B" will be equal, while the force in "C" will be zero.

$$V_A = V_B = \frac{P}{nL}(H - \frac{h_o}{2})$$

4.3.4 Case 4

Circular group of columns

Below fig (4.4) shows the tower subjected to a wind force" on the water tank.

Let there are "n" columns arranged symmetrically on a circle of radius "r". The other figure shows the plan. The whole framework will have a tendency to rotate about the axis of bending perpendicular to the direction of wind. Let$V_o, V_a, V_b, V_r\ etc.$ Be the axial forces in the columns situated at distances o, a, b,r. from bending axis. Due to wind moment "PH" at the column base the axial loads are related as follows.

$$V_b = V_r\frac{b}{r}$$

$V_a = V_r\frac{a}{r}$ $\qquad\qquad\qquad$ $V_o = 0$

If the columns are assumed to be hinged at the bottom the external moment $M_W = PH$ will be equal to the moment of resistance M_R

$$M_W = PH = M_R = 2V_r(r) + 4V_b(b) + 4V_a(a)$$

For generalized treatment consider a staging having "n" number of columns and area of each column is "A" subjected to the wind movement "M_w ". Therefore the columns are subjected to the compression in addition to the gravity load. The additional compression on the columns that is on the leeward side is proportional to the distance between the column and the axis of bending. For simplification of the analysis let us replace the staging configuration with equivalent ring of thickness "T"

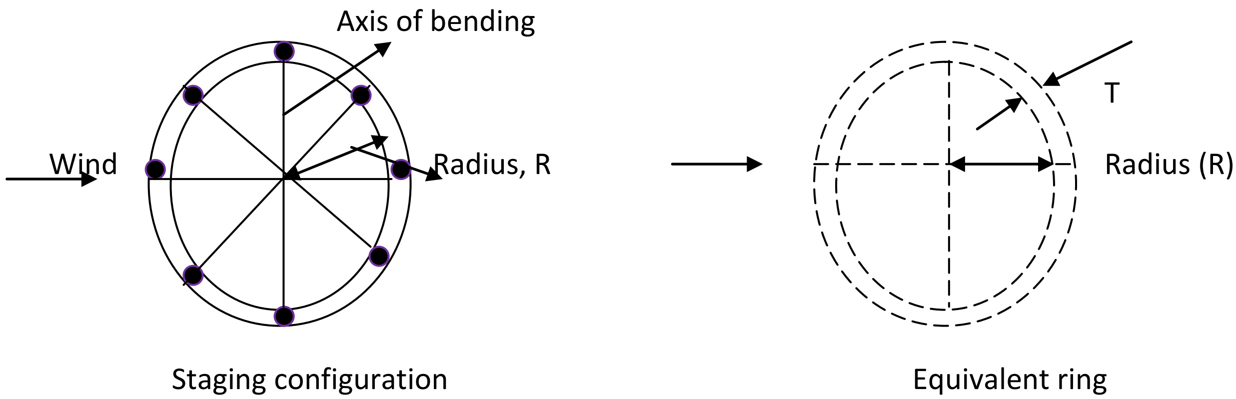


Fig.4.4.Columns arranged symmetrically on a circle of radius "R"

"n" columns total area= nA

Thickness of equivalent ring= T= $\frac{nA}{2\pi R}$

Second moment of area of ring about its diameter = $\pi R^3 T$

Therefore bending stress $f_b = \frac{M_w}{I} R = \frac{M_w R}{\pi R^3 T} = \frac{M_w}{\pi R^2 T}$

Where "R" is the Radius of column circle.

Therefore the force due to wind in leeward column=$P_w = \frac{M_w A}{\pi T R^2}$

Substituting the value of thickness "T"$P_w = \frac{M_w A}{\pi R^2}\left(\frac{2\pi R}{nA}\right) = \frac{M_w}{\frac{1}{2} n R}$

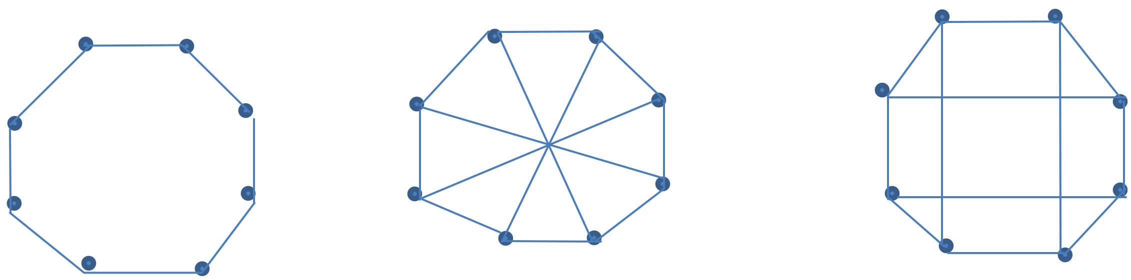
For example number of columns are = 4, therefore $P_w = \frac{M_w}{\frac{1}{2}(4)R} = \frac{M_w}{2R}$

4.4. Bracing

Horizontal bracings shall be provides if the height of the staging is more than 6m above the foundation for connecting the columns rigidly by suitably spacing vertically at a spacing not exceeding 6m.If the horizontal forces act in the critical direction bending moments in horizontal braces shall be calculated. Therefore the final moments in braces shall be the sum of in the lower and upper columns at the joint resolved in the direction of horizontal forces.

Two different supporting systems with basic supporting system

1) Radial bracing and
2) Cross bracing



Basic staging pattern staging with radial bracing staging with cross bracing

Fig.4.5. different patterns of the staging

4.5. Force in braces: Transverse shear

It is assumed that horizontal forces caused by the wind load is equally distributed on the top of the columns, the force on each column is$\frac{P_H}{n}$. This force causes different effects in the horizontal and diagonal braces thatare in horizontal braces compressive forces and in diagonal braces tensile forces. They are specifying the two different cases of wind direction for 6-column staging. Therefore by resolving the horizontal forces ($\frac{P_H}{n}$) in their planes the transverse shear in each horizontal shear could be found. Therefore among the two cases the maximum shear (= $2\frac{P_H}{n} = \frac{P_H}{3}$) occurs in case (b), this is because the braces support the columns laterally therefore an additional 2.5% of the column load is taken as shear in the panel. Thus total transverse shear "Q" or the total horizontal force will be equal to +2.5% of ($\frac{W+P_W}{n}$)

Chapter 5
Joints

5.0. Introduction

Joints are potential sources of leakage therefore all the joint must be water tight. Design of ordinary R.C.C structures is different from liquid retaining structures as the liquid retaining structures requires that concrete should not crack and hence concrete should subject to the tensile stresses which are within permissible limits. Therefore in various elements design of the water tank particularly in the tank portion the stresses have to be checked whether they are within the permissible limits or not.

A reinforced concrete member of liquid retaining structures is designed on the usual principles ignoring tensile resistance of concrete in bending. Cracking may be caused due to restraint to shrinkage expansion and contraction of concrete due to temperature or shrinkage and swelling due to moisture effects. Such restraint may be caused by

1) The interaction between concrete and reinforcement during shrinkage due to drying
2) The boundary conditions
3) The differential conditions prevailing through the large thickness massive concrete.

Therefore the above said effects can be overcome by certain measures like use the smaller size bars placed properly leads to closer cracks but of smaller width. Particularly the risk of cracking due to temperature and shrinkage effects may be minimized by limiting the changes in moisture content and temperature to which the structure as a whole is subjected. In case the length of the structure is large it should be subdivided into suitable lengths separated by movement joints. Especially where sections are changed the movement joints should be provided.

Movement joints and Construction joints must be properly detailed using quality water stops. Badly designed and detailed may permit flow of liquid and shall be avoided during design.

The special considerations required are as follows for reinforced concrete liquid retaining structures.

i) The concrete should be durable, impervious and maintenance free. Durability includes resistance to damage and protection against corrosion of reinforcements.
ii) In order to prevent leakage concrete leakage cracking in concrete shall be limited. Concrete has numerous cracks. Large crack width permits leakage of liquids and shall be restricted. Therefore two types of cracks shall be given attention.

a) Cracks due to shrinkage and temperature: These cracks are uniform throughout the depth of concrete. Such cracks can be limited by resisting the shrinkage and temperature forces by reinforcement.

b) Cracks due to applied loads: These cracks are wider on the surface and can permit water which may corrode the reinforcement and finally may lead to disintegration of concrete.

   Surface cracks shall be limited to predetermined values as suggested by respective codes of practice.

Is. 3370-2009 stipulates for liquid retaining structures the exposure as severe and permits crack width up to 0.2 mm. This requirement necessitates limiting tensile stresses in concrete to permissible value.

The design method developed considering limiting crack width is known as “no crack” design or uncrack theory.

5.1. Common Joints in water tanks

The various types of joints may be categorized under 3 –heads.

1) Movement joints
2) Construction joints
3) Temporary open joints

5.1.1. Movement joints

These joints require special materials is to be incorporated in order to maintain water tightness in accommodating relative moment between the sides of the joints. Therefore all movement joints are essentially comes under flexible joints. Movement joints are of 3-types.

a) Contraction joints
b) Expansion joints
c) Sliding joints

5.1.1.1. Contraction joints

It is a type of typical movement joint which accommodates the contraction of the concrete.

The joint may be either a partial contraction joint in which there is discontinuity of concrete but the reinforcement run through the joint or complete contraction joint in which there is discontinuity of both concrete and steel. In both the cases no initial gap is kept at the joint but only discontinuity is given during construction .In the former type the mouth of the joint is filled with joint sealing compound and then strip painted while in the later type a water bar is inserted. A water bar is a pre formed strip of impermeable material (such as a material, polyvinyl chloride or rubber.)Joint sealing compounds are impermeable ductile materials which are required to provide a water tight seal by adhesion to the concrete throughout the range of joint movement.

5.1.1.2. Expansion joint

It is a movement joint with complete discontinuity in both reinforcement and concrete and is intended to accommodate either contraction or expansion of the structure. In general such joint requires the provision of an initial gap between the adjoining parts of a structure which by closing or opening accommodates the expansion or contraction of the structure. The initial gap is filled with joint filler. Joint fillers used are usually compressible sheet or strip materials used spacers.

5.1.1.3. Sliding joint

Sliding joints is a type of movement joint with complete discontinuity in both concrete and reinforcement at which special provision is made to facilitate relative movement in place of the joint. A typical application of such joint is between floor and wall in some cylindrical tank designs.

5.1.2. Construction joints

A construction joint is a type of joint in the concrete introduced for convenience in construction at which special measures are taken to achieve subsequent continuity without provision for further relative movement. It is therefore a rigid joint in contrast to a movement joint which is a flexible joint.

Therefore the position and arrangement of all construction joints should be predetermined by the engineer. Consideration should be given to limiting the number of such joints and to keeping them free from possibility of percolation in a manner similar to contraction joints.

CHAPTER 6

COLUMN FOUNDATION

6.1. Introduction

The design of foundation and the forces on various elements of the tank because of wind loads affected by the type of soil at foundation therefore Geotechnical investigation of the site is sincerely required particularly the differential settlement effects should be properly taken into account. At foundation level in order to determine the soil properties minimum 10 m deep bore holes of 150 mm diameter shall be taken. If corrected "N" value (Standard penetration Test) is less than 15, therefore the alteration is the ground shall be properly compacted to achieve N> 15 and other measures shall be taken.

The selection of a particular type of foundation is often based on a number of factors. Such as adequate depth to prevent frost damage, bearing capacity, settlement, quality, adequate strength, adverse soil changes and wind forces. Based on the analysis of all the factors listed above specific type of foundation would be recommended based on soil exploration by engineer.

Separate footings may be provided for column staging and designed as per requirements of EBCS-7, combined footing with or without tie beam or raft foundation in accordance with EBCS-7 may be provided.

The foundation shall be so designed and proportioned that under both gravity loads of tower(with tank full as well as empty) and effects due to horizontal forces the pressure caused by these on the soil is within the safe bearing capacity and the footing in the critical direction does not lift up at any point. Loss of contact between footing and underneath soil should not be allowed .Loss of contact may be allowed in locations where the Safe bearing capacity is high provided it is safe against overturning and such other considerations that are to be fulfilled.

Based on soil types in Ethiopia which are basically of expansive type mat foundations are very much suitable. And also the wind load may act in any direction therefore it's effect on the foundation is not uniform in order to keep the stresses within the permissible limits and distribute the loads to the lateral load (i.e., wind load) resisting system(staging) uniformly if the foundation is of slab type which will act as a diaphragm. From economic considerations mat foundations are often constructed because of the following reasons:

1) Large individual footings: It is in the case when the sum of individual footing areas exceeds about one half of the total foundation area a mat foundation is often constructed

2) Cavities or compressible lenses: when the subsurface exploration indicates that If there will be unequal settlement caused by small cavities or compressible lenses below the foundation a mat foundation can be used.
3) Shallow settlements: A mat foundation can be recommended when the mat foundation would minimize differential settlements and shallow settlements predominate.
4) Unequal distribution of loads: In the case of some structures loads acting on different areas of the foundation can have large difference in building loads. A mat foundation would lend to distribute the unequal building loads and reduce the differential settlements. Because the conventional spread footings could be subjected to excessive differential settlement.
5) Hydrodynamic uplift: Due to a high groundwater table the foundation will be subjected to hydrostatic uplift, In order to resist the uplift forces a mat foundation could be used to resist

6.1. Problem statement: Analyze and design the Intze water tank form the wind load point of view. The site is located in the urban center with a zero altitude. For different capacities with varying staging height.

Solution: In order to differentiate how the wind load effect is varying with the variable height of the staging. The sizes of the tanks are chosen 1200m3 and 1600m3 capacities. The staging height is varying with 4m difference with 12m to 28m height. For analyzing the wind load on the tank portion various provisions of the EBCS-1, wind load calculations are taken.

6.2. Wind data

Terrain category: Zone IV (according to the EBCE-1, Table 3.2 Terrain categories and related parameters)

K_T - Terrain factor =0.24, Z_o (m) - roughness length = 1 and z_{min}(m)- Minimum height =16

Wind velocity: Basic mean reference wind velocity $V_{O, ref}$= 22 m/sec.

Wind load calculation:

Pressure coefficients for the roof and bottom of the tank should be calculated. External pressure coefficient is based on the exposure coefficient in this the variable is roughness coefficient which depends upon the reference height. For the domes according to the EBCS-1 A.2.8 given the reference height is equal to the “h+ f/2”. For finding the pressure coefficients Fig. A.9 and A.10 should be used. For our case we are using the bottom is circular therefore the fig A.10- External pressure coefficients C_{pe10}, for domes with circular base should be used. The table is based on the h/d ratio and f/d ratio. For example the “f “value is rise of the dome it is usually $(1/5)^{th}$ of the diameter of the tank, therefore for 12 m tank diameter the rise that is “f” is 2.4 meters. f/d ratio is 0.2 and h/d ratio for 24 meters height staging with cylindrical walls height of the tank is 2/3 times the diameter of the tank. Therefore for 9 m height cylindrical walls the ratio of h/d is 0.75, the $|C_{pe,10}$ is “-1.1”.The internal pressure c_{pi} inside the tank may be because of any liquid stored or in the case of water tanks if there is no pressure due to stored water inside the tank internal pressure will be generated due to small permeability, may be because of openings provide (which may small) at the roof level. Suppose if no openings exist, as in R.C.C water tanks C_{Pi}= 0

Usually roof pressure will be used with vertical loads for design of dome.

Therefore the overall horizontal Force on the Tank

On the top dome no horizontal force will act, because the load due to wind pressure on the dome has been included in the net vertical force associated with an eccentricity.

For finding the force on circular cylinders external pressure coefficients are taken based on the Reynolds number $R_e = \frac{bv_m(z_e)}{\vartheta}$, and the external pressure coefficients C_{pe} of circular cylinders are given by $c_{pe} = c_{p,o}\psi_{\lambda\alpha}$, and the external pressure coefficient $c_{p,o}$ is given in fig A.22 for various Reynolds number as a function of angle "α" and the reference area A_{ref} is = lb. Also the reference height is considered is equal to the height above the ground of the section being considered. For conical bottom also to be considered similarly. In the finding of the pressure on these elements average wind pressure consideration should be taken. That is for that element top height considered and calculated the roughness coefficient find the exposure coefficient and then finally the reference wind pressure like this the bottom height should be taken and all the parameters to find the reference wind pressure shall be calculated.

6.3. Staging

In order to calculate the wind force on columns, it is required to consider each column as individual member and no shielding effect is considered on columns located on leeward side as the columns are placed far apart on periphery only.

In designing the columns the load on the columns is due to container (with tank full), self-weight of column and weight of the bracing. Therefore the weight on each of the column is total load over number of columns in staging. Thus the following loading cases have to be considered in the column design. And also bracing is used for increasing the stiffness of the vertical members which resist the later load and also in order to reduce the bending and shear in the columns. The design of the columns is done by using the limit state method. Therefore the columns must be checked form the following loading cases.

i) D.L + L.L
ii) D.L + L.L +W.L , when tank is empty on wind ward column
iii) D.L + L.L +W.L , when tank is full on wind ward column
iv) D.L + L.L +W.L , when tank empty on Lee ward column
v) D.L + L.L +W.L , when tank full on Lee ward column

Wind action is represented either as a wind pressure or a wind force. The action on the structure caused by the wind pressure is assumed to act normal to the surface except where otherwise specified. e.g., for tangential friction forces.

Calculation of the pressure

External pressure:

The wind pressure acting on the external surfaces of a structure, we shall be obtained from$W_e = q_{ref} c_e(Z_e) c_{pe}$

c_{pe} –Internal pressure coefficient

Internal Pressure:

The wind pressure acting on the internal surfaces of structure $W_i = q_{ref} c_e(z_i) c_{pi}$

c_{pi} – Internal pressure coefficient

Net pressure:

The net wind pressure across a wall or an element is the difference of the pressures on each surface taking due account of their signs (pressure directed towards the surface is taken as positive and suction directed away from the surface as negative.

6.4. Wind forces from pressures

The wind forces acting on a structure or a structural component may be determined in two ways

a) By means of global forces
b) As a summation of pressures acting on surfaces provided that the structure or the structural component is not sensitive to dynamic response($c_d < 1.2$)

The global force F_w shall be obtained from the following expression:

$$F_w = q_{ref} c_e(z_e) c_d c_f A_{ref}$$

c_f —Force coefficient

A_{ref} —Rreference area forc_f(generally the projected area of the structure normal to the wind)

The following parameters are used several times are defined below:

q_{ref} —Rreference mean wind velocity pressure derived from reference wind velocity. It is used as the characteristic value

$c_e(z)$ —Exposure coefficient accounting for the terrain and height above the ground "Z". The coefficient also modifies the mean pressure to a peak pressure allowing for turbulence

Z- Reference height appropriate for the relevant pressure coefficient (Z=z_e)for external pressure and force coefficient, (Z=Z_i) for internal pressure coefficient.

For this case though it is cantilevered structure with a slenderness ratio $^{height}/_{width} < 2$ the force F_{wj} — is not to be calculated.

Reference wind

The reference mean wind velocity pressure "q_{ref}" shall be determined from

$$q_{ref} = \frac{\rho}{2} {v_{ref}}^2$$

v_{ref} —is the reference wind velocity

ρ —is air density

The air density is affected by altitude and depends on the temperature and pressure to be expected in the region during wind storming. A temperature of 20^0 has been selected as appropriate for Ethiopia and the variation of mean atmospheric pressure with altitude is given below.

Values of air density ρ

Site altitude (m) above sea level	$\rho\ {kg}/{m^3}$
0	1.20
500	1.12
1000	1.06
1500	1.00
2000	0.94

Table 6.0. Value of Air density

Reference wind velocity

The reference wind velocity "v_{ref}" is defined as the "10" minute mean wind velocity at "10m" above ground of terrain category-II having an annual probability of exceedence of 0.02(commonly referred to as having a mean return period of 50 years.

It shall be determined from

$$v_{ref} = c_{DIR}\ c_{TEM}\ c_{ALT}\ v_{ref,o}$$

c_{DIR} —is the direction factor to be taken 1.0

c_{TEM} —is the temporary (seasonal) factor to be taken as 1.0

c_{ALT} —is the altitude factor as 1.0

$v_{ref,o}$ —is the basic value of the reference wind velocity to be taken as 22m/sec.

6.5. Roughness coefficient

The roughness coefficient $c_r(z)$ accounts for the variability of the mean wind velocity at the site of the structure due to

1) The height above ground level
2) The roughness of the terrain depending on the wind direction.

The roughness coefficient at height "Z" is defined by the logarithmic profile:

$$c_r(z) = k_T l_n\left(\frac{Z}{Z_o}\right) for\ Z_{min} \leq Z \leq 200mm$$

$$c_r(z) = c_r(z_{min}) for\ Z < Z_{min}$$

k_T —Tterrain factor

Z_o —Rroughness length

Z_{min} —Minimum height

Table 3.2 of EBCS-1 is providing terrain categories and related parameters.

6.6. Topography coefficient

The topography coefficient $c_r(z)$ accounts for the increase of mean wind speed over isolated hills and escarpments (not undulating and mountainous regions). It is related to the wind velocity at the base of the hill or escarpment. It shall be considered for locations within topography affected zone.

$$c_t = 1 \, for \;\; \emptyset < 0.05$$

$$c_t = 1 + 2S\emptyset \quad for \; 0.05 < \; \emptyset < 0.3$$

$$c_t = 1 + 0.6S \quad for \; \emptyset > 0.3$$

6.7. Exposure coefficient: For codification purposes it has been assumed that the quasi static gust load is determined from

$$c_e(Z) = c_r(z)^2 c_t(z)^2 [1 + \frac{7k_T}{c_r(z)c_t(z)}$$

k_T —Terrain factor

$c_r(z)$ —is the roughness coefficient

$c_t(z)$ —is the topography coefficient

Wind load on various elements at 24 m height of staging for 1600m3 tanks

No	Description	Wind load in KN	Height from the ground level	Moment at the base
1	Top Dome	16.48	38.0	626.24
2	Cylindrical wall	76.38	31.5	2405.97
3	Conical Dome	12.72	26.1	331.992
4	Columns 12 no's	170.1	12	2041.2
5	Bracings	28.5	11.4	324.9
Total		$\sum H_w$ =304.18 Kn		$\sum M_w$ =5730.3 KN-m

Table6.1. Wind load on various elements

Details of the sizes of the of the members for 1200 m3 and 1600 m3 capacity tanks with a staging height is 24m.

No	Item description	Tank 1200m3	Tank 1600m3
1	Top Dome	100 mm	130mm
2.	Cylindrical wall	200 mm at top and 350 mm at bottom	200 mm at top and 450 mm at bottom
3.	Top Ring beam	400 x 400 mm	500 x 500 mm
4.	Middle ring beam	1200 x 600	1200 x 700 mm
5.	Conical dome	550 mm	650 mm
6	Bottom dome	250 mm	330 mm
7.	Bottom ring girder	600 x1200	700 x1200 mm
8	Column	700 mm	800 mm
9.	Bracing	500 x 500	550 X 550 mm
10	Raft foundation.	600 mm thick slab	680 mm thick slab

Table 6.2.Sizes of the various members

For 24 m height staging the capacity of the tanks 1200m3 and 1600m3 reinforcement details are shown below.

No	Description	Capacity 1200m^3	Capacity 1600m^3
1	Top dome Main and distribution	ϕ8 mm c/c 140mm both ways.	ϕ8 mm c/c 90 mm both circumferentially and meridionally
2	Top Ring beam B1 Main Stirrups	i) 12 ϕ16mm ii) ϕ 8mm two legged c/c 150 mm	i)16 ϕ16 mm ii) ϕ 8 mm two legged c/c 150 mm
3	Vertical wall Main hoop steel- from top i) 0 to 2m ii)2 to 4m iii) 4 to 9m	i) ϕ12mm c/c180 mm on both sides. ii) ϕ20 mm c/c250 mm iii) ϕ25 mm c/c 150 mm	i) ϕ12 mmc/c 90 mm both sides ii) ϕ20 mm c/c 110 mm iii) ϕ32 mm c/c100 mm

	Distribution - From top i) 0 to 2m ii) 2 to 4m iii) 4 to 9m	i) ϕ12 mmc/c 275 mm ii) ϕ12 mm c/c 150 mm iii) ϕ 12 mmc/c 110 mm	i) ϕ12mm c/c 190 mm ii) ϕ12 mm c/c 100 mm iii) ϕ 12 mm c/c 90 mm
4	Bottom ring beam B2 i) Main ii) Stirrups	 i) 24ϕ20 mm ii) ϕ 10 mm c/c 100 mm	 i) ϕ25 mm c/c 22 mm ii) 12 mm c/c 100 mm
5	Conical wall i) Main ii) Distribution	 i) ϕ25 mmc/c 150 mm ii) ϕ 12 mm c/c 110 mm	 i) ϕ36 mm c/c100 mm ii) ϕ16 mm c/c 100 mm
6	Bottom spherical dome	iii) ϕ12 mm c/c100 mm both sides	iii) ϕ16 mm c/c100 mm both sides
7	Bottom circular girder(B3) i)Main top ii) Main bottom iii) Vertical stirrups	 i) 16 ϕ25 mm ii) 8 ϕ 25 mm iii) ϕ 12 mm Six legged c/c 200 mm	 i) 22 ϕ25 mm ii)10 ϕ 25 mm iii) ϕ 12 mm ϕ six legged 140 mm c/c
Supporting tower: Staging			
1	Column i) Main ii) Lateral ties.	 i) 10 ϕ 36 mm ii) ϕ 12 mm c/c 250 mm	 i) 16 ϕ 36 mm ii) ϕ 12 mm c/c220 mm
2	Bracing i) Main ii) Stirrups	 i) 6 ϕ 25 mm at top and bottom ii) ϕ 12 mmc/c 250 mm	 i) 8 ϕ 25 mm at top and bottom ii) ϕ 12 mm c/c 200 mm
3	Circular girder for rafter i)Top ii)Bottom iii)Stirrups	 i) 6 ϕ 25 mm ii) 10 ϕ 25 mm iii) ϕ 12 mm c/c90 mm	 i) 6 ϕ 36 mm ii) 8 ϕ 36 mm
4	Raft foundation i) Main ii) Distribution	 i) ϕ 25 mm c/c150 mm ii) ϕ 12 mm c/c120 mm	 i) ϕ 25 mm c/c120 mm ii) ϕ 12 mm c/c90 mm

Table 6.3. Steel Requirement

Wind load variation with respect to the height wise variation of staging from 12m to 24 m also variation of the capacity of the 1600 m3 tank.

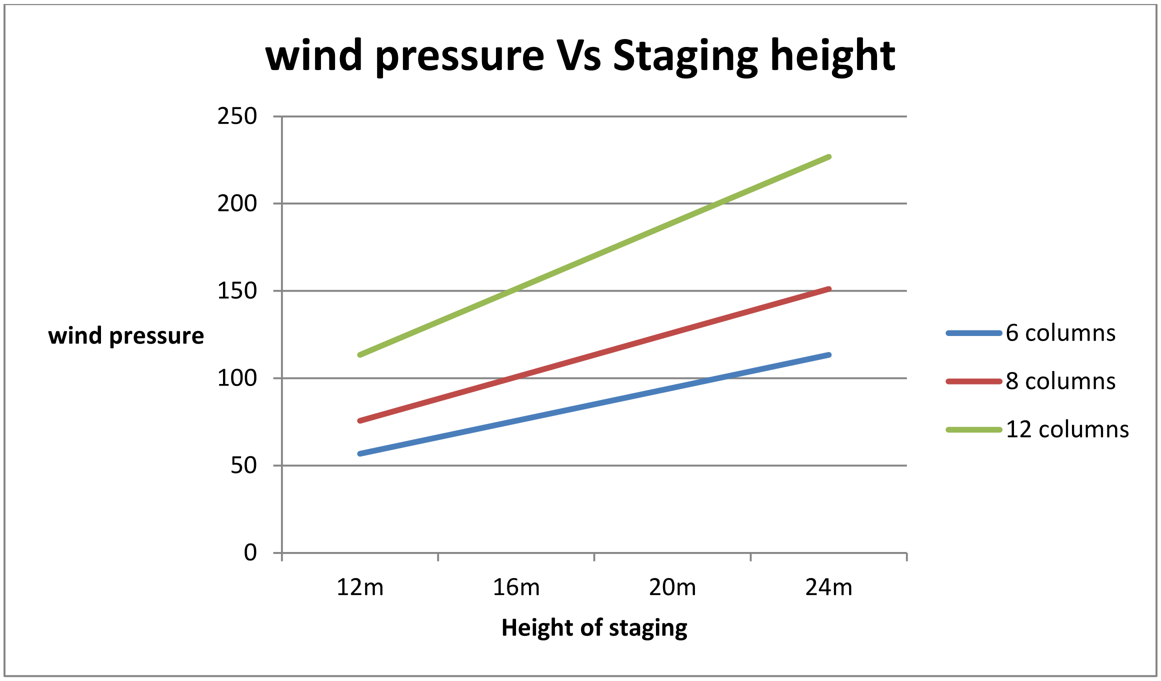


Fig.6.1. (a) Wind pressure Vs Staging height

For 12 columns and the staging height is 24m the variation of wind force on the 1200m3 and 1600 m3 water tank.

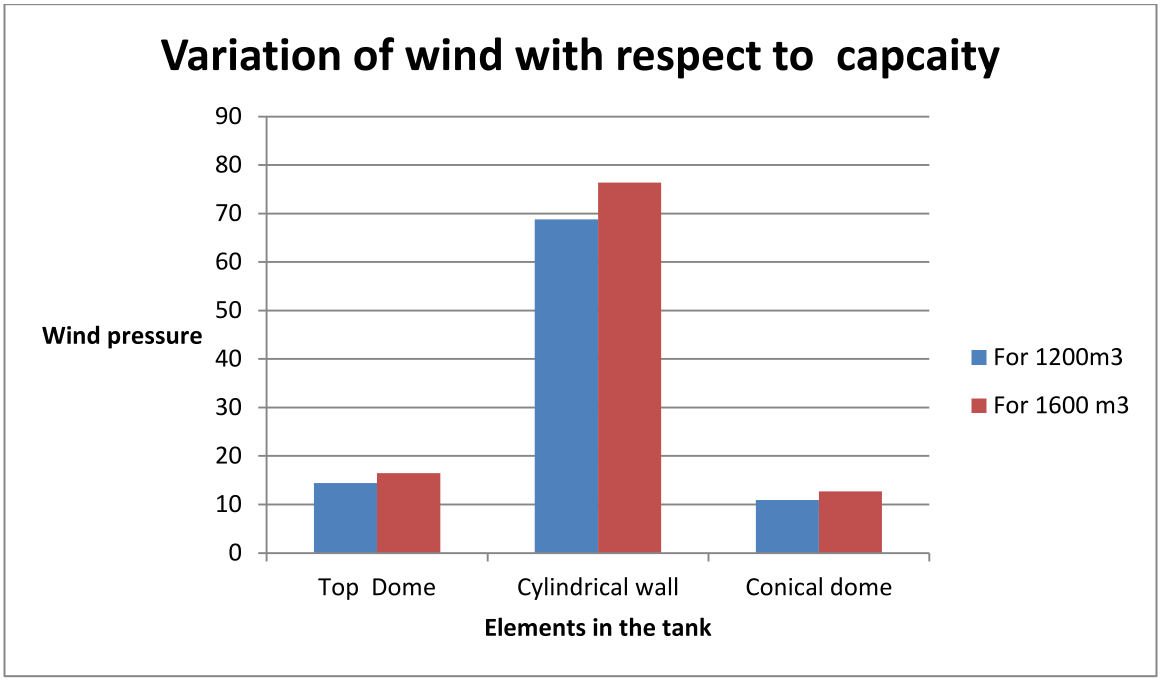


Fig.6.2. (b) Variation of wind with respect to capacity

CHAPTER-7

EARTHQUAKE

7.0. Introduction

One of the major problems that may lead to failure of elevated concrete water tanks is earthquake. Therefore the analysis of elevated tank must be carefully performed so that safety can be assured when earthquake occurs and the tanks remain functional even after earthquake. The irregular shape of the elevated water tanks for which most of the mass concentrated in the upper part of the tank makes it more sensitive to any dynamic load especially due to an earthquake. The elevated tanks are subjected to lateral and torsional vibrations due to wind and seismic forces. These lateral forces physically induce two different types of vibration in the water of the tank. Due to this vibration water exerts impulsive and convective hydrodynamic pressure on the tank wall and tank base in addition to the hydrostatic pressure. The effect of impulsive and convective hydrodynamic pressure should consider in the analysis of tanks. For small capacity tanks, the impulsive pressure is always greater than the convective pressure but it is vice versa for tanks with large capacity. Magnitudes of both the pressure are different , A part of water at the upper portion of the tank participate in sloshing motion (convective) with a longer period while the rest of the water at the bottom portion of the tank experiences the same impulsive vibration as the tank container is rigidly attached with container wall.

Basically there are three cases that are generally considered while analyze the elevated water tank

1) Empty condition
2) Partially filled condition
3) Fully filled condition

For (1) and (3) case the tank will behave as a one mass structure and for (2) case the tank behave as two mass structure. If we compared the case (1) and (3) with case (2) for maximum EQ force the maximum force to which the partially filled tank is subjected may be less than half the force to which the fully filled tank is subjected

Most elevated water tanks are never completely filled with water. Hence a two mass idealization of the tank is more appropriate as compared to one mass idealization

Analysis and design of elevated water tanks against earthquake effect is of considerable importance. This structure must remain functional even after an earthquake. Elevated water tanks which typically consist of a large mass particularly susceptible to EQ damage. Thus analysis and design of such structure against the EQ effect is of considerable importance. The following points are to be considered at the time of seismic analysis of elevated water tanks.

1) Elevated water tanks are vulnerable to EQ excitation mainly because of the relatively small resistance that the supporting system can offer during seismic events.
2) The seismic analysis and design of liquid storage tanks are complicated by many number of problems for examples: Dynamic interaction between contained fluid and vessel which is called fluid – structure interaction, sloshing motion of the contained fluid and dynamic interaction between vessel and supporting structure. In addition the supporting tower may need to be analyzed in post elastic state and for special cases a 3-dimensional analysis may be required to take into torsional effect on the supporting structure.
3) Tanks that are inadequately designed and detailed have suffered extensive damage during past earthquake. Knowledge of pressures and forces acting on the walls and bottom of containers during an earthquake and frequency properties of containers is important for good analysis and design of EQ restraint structures/facilities.

A simplified analysis procedure has been suggested by Housner in 1963 for fixed base elevated tanks. In this approach the two masses ($m_i - impulsive\ mass\ and\ m_c - convetive\ mass$) are assumed to be uncoupled and the EQ forces on the support are estimated by considering two separate single degree of freedom systems. The mass m_c represents only the sloshing of the convective mass; the mass consists of the impulsive mass of the fluid the mass derived by the weight of the container and by some parts self-weight of the supporting structure. This two masses model suggested by Housner has been commonly used for seismic design of elevated tanks. Similar equivalent masses and heights for this model based on the work of Velestos and co-workers (Malhotra) with certain modification that the procedure simple are also suggested in the Euro code-8(EC-8)

The total seismic response of a tank structure should be analyzed in terms of natural periods of vibrations, base shear force and over turning moments. Periods are necessary after determination of the two masses of $m_i\ and\ m_c$ with their locations and stiffness's. Base shear andoverturning moment for design can be estimated using standard structural dynamic procedures. It should be noted that concrete and steel tanks show different behavior under a seismic action. In the case of concrete tanks the wall may be taken as rigid whereas in the case of steel tanks the wall may be taken as flexible.

Parameters of spring mass model (i.e.$m_i m_c\ h_i h_i' h_c h_c' k_c$) are available for circular and rectangular tanks only. For tanks other shapes equivalent circular tank is to be considered Joshi (2000) has shown that such an approach gives satisfactory results for Intze tanks. Euro code-8 has suggested equivalent circular tank approach. And for tank shapes other than circular and rectangular (like Intze and truncated conical shape) the value of $\frac{H}{R}$ shall correspond to that of an equivalent circular tank of same volume and diameter equal to diameter of tank at top level of liquid and $m_i m_c \ h_i h_i{}' h_c h_c{}' k_c$ of equivalent circular tank shall be used.

A) The natural period of the impulsive mode of vibration T_i in second for elevated tank is $T_i = 2\pi\sqrt{\frac{m_i + m_s}{k_s}}$ where,

m_s —Mass of container and 1/3 mass of staging

m_i – Impulsive mass (The impulsive and convective masses m_i *and* m_c are given in Table B.1 as fractions of the total liquid mass "m"

k_s —Lateral stiffness of staging

Lateral stiffness of the staging (k_s) is the horizontal force required to be applied at the Centre of gravity of the tank to cause a corresponding unit horizontal displacement. The flexibility of bracing beam shall be considered in calculating the lateral stiffness k_s of elevated moment resisting frame type tank staging. For elevated tanks with moment resisting type frame staging the lateral stiffness can be evaluated by computer analysis or by simple procedures (Sameer and Jain 1992) or by established structural analysis method.

Lateral stiffness of staging is defined as the force required to be applied at the C.G of tank so as to get a corresponding unit deflection. C.G of the tank is the combined C.G of empty container and impulsive mass. However in this example C.G of tank is taken as C.G of empty container.

Natural periods given by EC-8 for impulsive mode $T_i = C_i \frac{H\sqrt{\rho}}{\sqrt{E}\sqrt{\frac{S}{R}}}$

ρ —Mass density of the liquid

E – Young's modules of elasticity of tank material

The coefficientsC_i *and* C_c) are obtained from Table B.1

S – Equivalent uniform thickness of the tank wall

S – For tanks with non-uniform wall thickness "S" may be computed by taking a weighted average over the wetted height of the tank wall, assigning highest weight to the thickness near the base of tank where the strain is maximum.

B) The natural period of the convective mode vibration T_c in seconds $T_c = C_c\sqrt{R}$ where R-in meters.

C_c —From Table B.1 of Ec-8 part-4.

C) Total base shear at the bottom of staging is given $V = \sqrt{{V_i}^2 + {V_c}^2}$

D) Total over turning moment at base of staging is given $M' = \sqrt{M_i{}' + M_c{}'}$

For elevated tank staging components should be designed for the critical direction of seismic force .Different components of staging may have different critical directions. For elevated tanks supported on frame type staging the design of the staging member should be for the most critical direction of horizontal base acceleration. For a staging consisting of four columns horizontal acceleration in diagonal direction (i.e.$45^0\ to\ Y - direction$) turns out to be most critical for axial force in columns. For brace beam most critical direction of loading is along the length of the brace beam. Sameer and Jain (1994) have discussed in detail the critical direction of horizontal base acceleration for frame type staging.

Problem: Analysis and Design of Elevated Intze water tank from the seismic forces.

Solution : for this problem the dimensions that were derived in the wind analysis for the Intze elevated tank of size 1600 m3 capacity with a staging height of 28 m , the diameter of the cylindrical wall is 12m and the height of the tank is 9m and the tank is filled with water to a height of 8m. The walls of the cylindrical portion of the tank are of varying thickness having four courses, each 2.25 m high. The lower most course is 450 mm thick and the next to that is 350 mm and the top most course is having 200 mm and below the top course the thickness is 250mm.The dimensions of the 1600 m3 capacity and 28 m staging height Intze tank from the wind analysis is

Element Type	Dimension
Top Dome	130 mm
Top Ring beam	500X500 mm
Cylindrical wall	200 mm @ top and 450 mm @ bottom
Bottom Ring beam	1200 X 700 mm
Circular Ring beam	1200 X 700 mm
Bottom Dome	330 mm
Conical Dome	650 mm
Braces	550X550 mm
Columns	Circular 800 mm dia.

Table7.0. Dimensions of the tank from the wind analysis

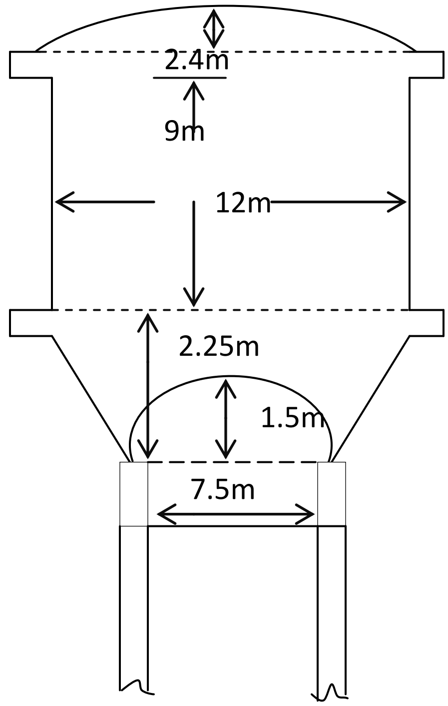


Fig.7.0. Dimensions of the various members

Weight Calculation:

Components	Calculation	Weight (KN)
Top Dome (130mm)	$r_1 - radius\ of\ the\ dome$ $= \left[\left(\left(\frac{12.5}{2}\right)^2 \Big/ 2.335 \right) + 2.335 \Big/ 2 \right] = 9.53m$ $= 2 * \pi * 9.53 * 2.335 * 0.13 * 25$	= 454.68
Top Ring beam	$\pi * (12 + 0.5)(0.5 * 0.5)25$	=245.5
Cylindrical wall	$\pi * (12 + \frac{(0.2 + 0.45}{2})) \frac{(0.2 + 0.45)}{2} * 9 * 25$	=2832
Bottom ring beam	$\pi * (12 + 1.2)(1.2 * 0.7)25$	= 871
Circular ring beam	$\pi * (7.5 + 1.2) * (1.2 * 0.7)25$	=574
Conical dome	$\pi * \left[\frac{\left(12 + \frac{0.45}{2}\right) + (7.5 + 1.2)}{2} \right] * 2.79 * 0.65 * 25$	=474.34
Bottom spherical dome	$\left(\left[\frac{(7.5+1.2)^2}{2} \Big/ 1.335 \right] + 1.335 \Big/ 2 \right) = 7.754\ m$ $2 * \pi * 7.754 * 1.335 * 0.33 * 25$	=536.84
Columns	$\pi * (0.8)^2 * \left(28 - \frac{0.7}{2}\right) * 12 * 25 \Big/ 4$	=4171
Braces	$1.96 * (0.55 * 0.55) * 6 * 7.5 * 25$	=668.47
Weight of the water	$1000 \left[\frac{\pi}{4}(12)^2 * 9 + \frac{\pi * 2.25}{12}(12^2 + 7.5^2 + 12 * 7.5 - \frac{\pi * (1.5)^2}{3}(3 * 7.754 - 1.5) \right]$	$=1137 \times 10^3$ Kg

Table 7.1.Weight of the elements of the tank

7.1. Height of the C.G of empty container above top of circular ring beam:

C.G of the empty container consists of: Top dome, top ring beam, cylindrical wall, bottom ring beam, bottom dome and circular ring beam.

$$\left[454.68*\left(2.25+9+\frac{2}{3}2.4\right)+245.5*\left(2.25+9-\frac{0.5}{2}\right)+2832*\left(2.25+\frac{2}{3}9\right)+871*\left(2.25-\frac{0.7}{2}\right)+474.34*\left(\frac{2}{3}2.25\right)+536.84\left(\frac{2}{3}1.5\right)-574*0.35\right]\Big/_{5988}$$

= 5.769m

Therefore the height of the empty container from top of footing = $24+\frac{0.7}{2}+5.769=30.119$m

Weight of the empty container = 454.68+245.5+2832.2+871+574+474.34+536.84= 5988 KN

Weight of staging = 4171+668.47= 4839.47 KN

Hence weight of the empty container +one third of the weight of the staging = $5988+\frac{4839.47}{3}=7601\ KN$

Model properties: First the equivalent uniform thickness of the tank wall is calculated by the weighted average method using weights equal to the distance from the liquid surface.

$$s=\frac{\mathbf{0.45*2.25*\left(8-\frac{2.25}{2}\right)+0.35*2.25*\left(8-2.25-\frac{2.25}{2}\right)+0.25*2.25*\left(8-2.25-2.25-\frac{2.25}{2}\right)+0.2*1.25*\frac{1.25}{2}}}{\mathbf{2.25*6.875+2.25*4.625+2.25*2.375+1.25*0.625}}$$

Therefore S= 0.3779m

For concrete E=$2X\ 10^5\ N/mm2$

$$\rho=1000\ Kg/m3$$

For obtaining parameters of spring mass model, an equivalent circular container of same volume and diameter equal to diameter of tank at top level of liquid will be considered. Let *H1* be the height of equivalent circular cylinder

$$\frac{\pi}{4}D^2*H1=Volume\ of\ the\ tank$$

$\frac{\pi}{4}(12)^2*H1=1137$ Therefore H1= 10m

For $\frac{H}{R}=\frac{10}{6}=1.667$ for This value from the table B.1

$\frac{H}{R}$	C_i	C_c	$\frac{m_i}{m}$	$\frac{m_c}{m}$	$\frac{h_i}{H}$	$\frac{h_c}{H}$	$\frac{h_i'}{H}$	$\frac{h_c'}{H}$
1.66.7	6.0975	1.48	0.7053	0.295	0.441	0.705	0.541	0.7415

Table 7.2. H/R ratio

Now calculate the Time period for impulsive and convective

$T_i = C_i \frac{H\sqrt{\rho}}{\sqrt{E}\sqrt{\frac{S}{R}}} = 6.06 \frac{8*\sqrt{1000}}{\sqrt{\frac{0.3779}{6}}\sqrt{2X10^{11}}}$ =0.0137 s

$T_c = C_c\sqrt{R} = 1.48\sqrt{6} = 3.62$ s

Hence

$m_i = 0.7053 * 1137X10^3$ =801X10^3 kg

$m_c = 0.295 * 1137X10^3$ = 335.415X10^3 Kg

$h_i = 0.441 * 10$ =4.41 m

$h_c = 0.705 * 10 = 7.05m$

$h_i{}' = 0.541 * 10 = 5.41m$

$h_c{}' = 0.7415 * 10 = 7.415m$

7.2. Seismic responses:

The impulsive spectral acceleration for $T\,_i = 0.137s$ obtain for 5% damped elastic response spectrum. The convective spectral acceleration for $T_c = 3.62\ s$ obtain for the 0.5% damped response spectrum .This is based on the ANNEX B (IINFORMATIVE) SEISMIC ANALYSIS PROCEDURES FOR TANKS. This Annex provides information on seismic analysis procedures for tanks subjected to horizontal and vertical excitation and having the following characteristics: a) cylindrical shape, with vertical axis and circular or rectangular cross-section; b) rigid or flexible foundation; c) fully or partially anchored to the foundation.$S_{e(T_i)}$ –The impulsive spectral acceleration obtained from a 2% damped elastic response spectra for steel or pre stressed concrete and a 5% damped elastic response spectrum for concrete tanks, $S_{e(T_c)}$ – Convective spectral acceleration obtained from a 0.5% damped elastic response spectrum.

For finding $S_{e(T_i)}$ and $S_{e(T_c)}$ according to the EBCS-8 which has given formulas based on the 5% damping curve for the elastic response spectrum, therefore application of damping correction factor to it is inevitable. The value of the damping correction factor “η" may be determined by the expression according to the new draft of the EBCS-8

$$\eta = \sqrt{\frac{10}{5+\xi}} \geq 0.55$$

η —is the damping factor with reference value $\eta = 1$ for 5% viscous damping.

ξ —is the viscous damping ratio of the structure expressed as percentage. Therefore for the 0.5% damping the correction factor is $\eta = \sqrt{\frac{10}{5+0.5}} = 1.348$

The structure is located on the Ground type is “B” according to the EBCS-8 ,Table 3.1 Ground types, the site soil satisfies as deposits of very dense sand, gravel or very stiff at least several tens of meters in thickness characterized by a gradual increase of mechanical property with depth. Therefore Table 3.2 values of the parameters describe the recommended Type-1 elastic response spectra to be used. From this table for type “B” soil the corresponding parameters are

$\boldsymbol{S = 1.35}$	$\boldsymbol{T_B(s) = 0.05}$	$\boldsymbol{T_c(s) = 0.25}$	$\boldsymbol{T_D(s) = 1.2}$

Table 7.3. Type-1 spectrum for “B” class soil

For finding horizontal response spectrum:

The EBCS-8 is providing formula for finding the horizontal components of the seismic action the elastic response spectrum is defined by the expressions given in the clause 3.2.2.2. For time periods for the impulsive and convective are $T_i = 0.0137\ s\ and\ T_c = 3.62\ s$, for these values the horizontal response spectrum values are

For impulsive time period $T_i = 0.0137\ s$, i.e, it satisfying the condition$0 \leq T_i \leq T_B$,$(0 \leq 0.0137 \leq 0.05)$ the equation for finding the horizontal elastic response spectrum is

$$S_e(T_i) = a_g * S * \left[1 + \frac{T}{T_B}(\eta * 2.5 - 1)\right]$$

Similarly for the convective time period $T_c = 3.62\ s$ ie, it is satisfying the relation $T_D \leq T_{Con} \leq 4\ s$ $(\ 1.2 \leq 3.62 \leq 4\ s)$, the equation for finding the horizontal elastic response spectrum is

$$S_e(T_{con}) = a_g * S * \eta * 2.5(\frac{T_C T_D}{T_{con}})$$

7.3. Fundamental requirement according to the EBCS-8:

The structure shall be designed and constructed to withstand the design seismic action without local or global collapse thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action is expressed in terms of a) The reference seismic action associated with a reference probability of exceedence P_{NCR} in 50 years or a reference return period T_{NCR} and b) the importance factor γ_I to take into account reliability differentiation. The values to be ascribed to P_{NCR} or T_{NCR} for or is in the National Annex document. The recommended values are $P_{NCR} = 10\%$ and $T_{NCR} = 475$ $years$. Therefore the importance factors given in EBCS code are related to the building structures. For the tanks the importance factor has taken from the EC-8, it has given based on the reliability, the classes defined according to this are three defined corresponding to situations with high (Class-1), medium (Class-2) and low(Class-3). Depending on the tank contents an importance factor γ_I is assigned to each of the three classes.

Importance factor (γ_I) for tanks according to EC-8.

Tank contents	**Importance factor (γ_I)**		
	Class-1	**Class-2**	**Class-3**
Drinking water, non-toxic, nonflammable chemicals	1.2	1.0	0.8
Firefighting water, non-volatile toxic chemicals lowly flammable petrochemicals.	1.4	1.2	1.0
Volatile toxic chemicals, explosive and higher flammable liquids	1.6	1.4	1.2

Table 7.4. Importance factor for tanks

7.4. Seismic zones: Clause 3.2 of EBCS-8

National territories shall be subdivided into seismic zones depending on its local hazard. By definition the hazard within each zone is assumed to be constant. For most of the applications of EBCS-8 the hazard is described in terms of single parameter i.e., the value of the reference peak ground acceleration on type “A” ground"a_{gR}". The reference peak ground acceleration on type “A” ground a_{gR} for use is derived from zonation maps found in the National Annex. The reference peak ground acceleration chosen for each seismic zone corresponds to the reference return period T_{NCR} of the seismic action for the no collapse requirement (or reference probability of exceedence in 50 yearsP_{NCR}). An importance factor γ_I equal to 1.0 is assigned to this reference return period. For return periods other than the reference the design ground acceleration on type “A” ground a_g is equal to a_{gR} times importance factor.($a_g = \gamma_I * a_{gR}$).

Therefore in the analysis according to the EC-8 elastic spectrum (Type-1, Soil type –B) was used as well as: reference peak ground acceleration $a_{gR} = 0.20g$ importance factor for the structural class -3 $\gamma_I = 1.20$, which gives design ground acceleration $a_g = 0.24g$.

Calculate the Horizontal response spectrum for the impulsive time period is

$$S_e(0.0137) = 0.24 * 9.81 * 1.35\left[1 + \frac{0.0137}{0.05}(1.348 * 2.5 - 1)\right] = 5.242$$

Similarly for the Convective time period the horizontal response spectrum is

$$S_e(3.62) = 0.24 * 9.81 * 1.348 * 2.5\left[\frac{0.25*1.2}{3.62}\right] = 0.6575$$

According to the EBCS-8
Fb = Sd (T1) m λ
Where,
Sd (T1) is the ordinate of the design spectrum at period T1;
T1 is the fundamental period of vibration of the building for lateral motion in the direction considered;
λ is the correction factor, the value of which is equal to: λ= 0.85 if T1 < 2 TC and the building has more than two story's, or λ = 1.0 otherwise
Therefore base shear at the bottom of the staging in impulsive mode is:
$F_i = S_e(T_i)(m_i + m_s)\ \lambda$, the value of the λ for the condition 0.0137< 2(0.25) is equal to 0.85

$$F_i = 5.242(801X10^3 + 760.1X10^3)0.85 = 6955.79X10^3 \text{ kg}$$

Similarly the base shear in the convective mode,

The λ is the correction factor for the condition 3.62 <2(0.25) is not satisfying therefore the correction factor λ= 1

$$\mathrm{F_c} = 0.6575(335.41\mathrm{X}10^3)*1 = 220\mathrm{X}10^3 \text{ Kg}$$

Total shear at the bottom of the staging =69557+2200 =71757 KN

Moment at the bottom:

Overturning moment at the base of the staging in impulsive mode

$$M_i = S_e(T_i)\big(m_i(h_i' + h_s) + m_s h_{cg}\big)$$

$= 5.242(801X10^3$(5.41+28.35) +760X10^3 $*$34.119) = 27768x10^3 kg-m=277.6 MNm

Similarly the overturning moment in convective mode is

$$M_c = S_e(T_{con})(m_c(h_c{'} + h_s)$$

$= 0.6575\big(335.41X10^3(7.415 + 28.35)\big)$ =7886 X10^3 kg-m=78.86 MNm

Total over turning moment = 277.6 + 78.86 =356.46 MNm

Comparison of the base shear and overturning moment for wind load and Earth quake load

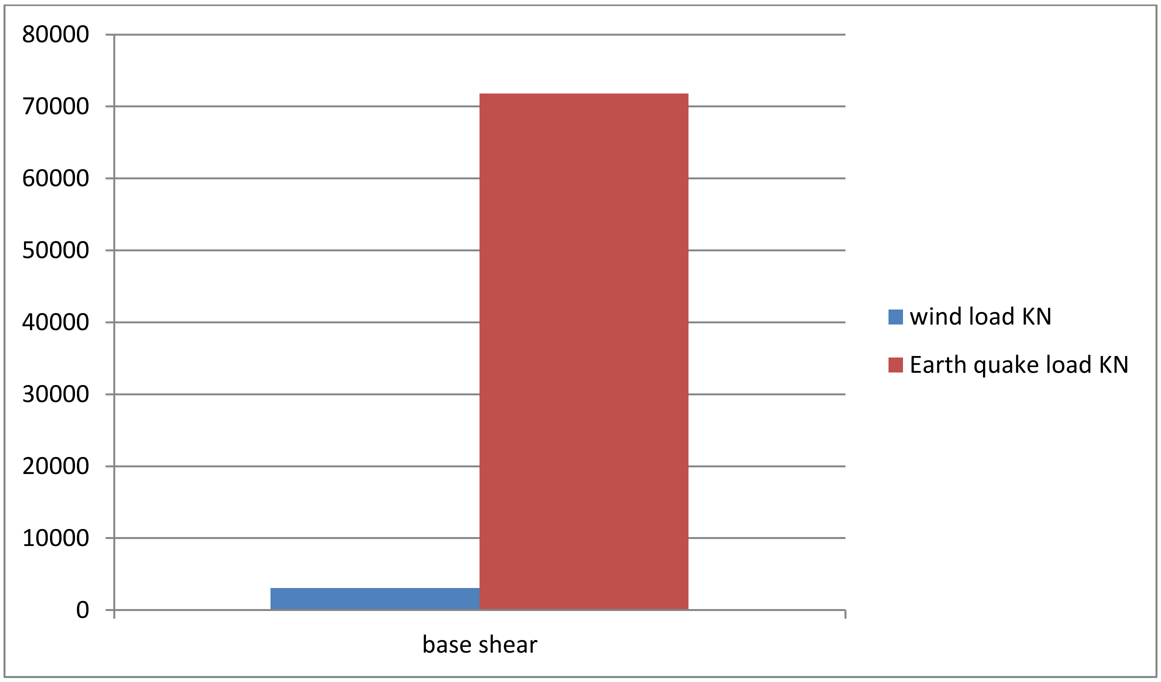


Fig.7.1. Base shear comparison for wind and earthquake

Comparison of the over turning moments for wind and earth quake load

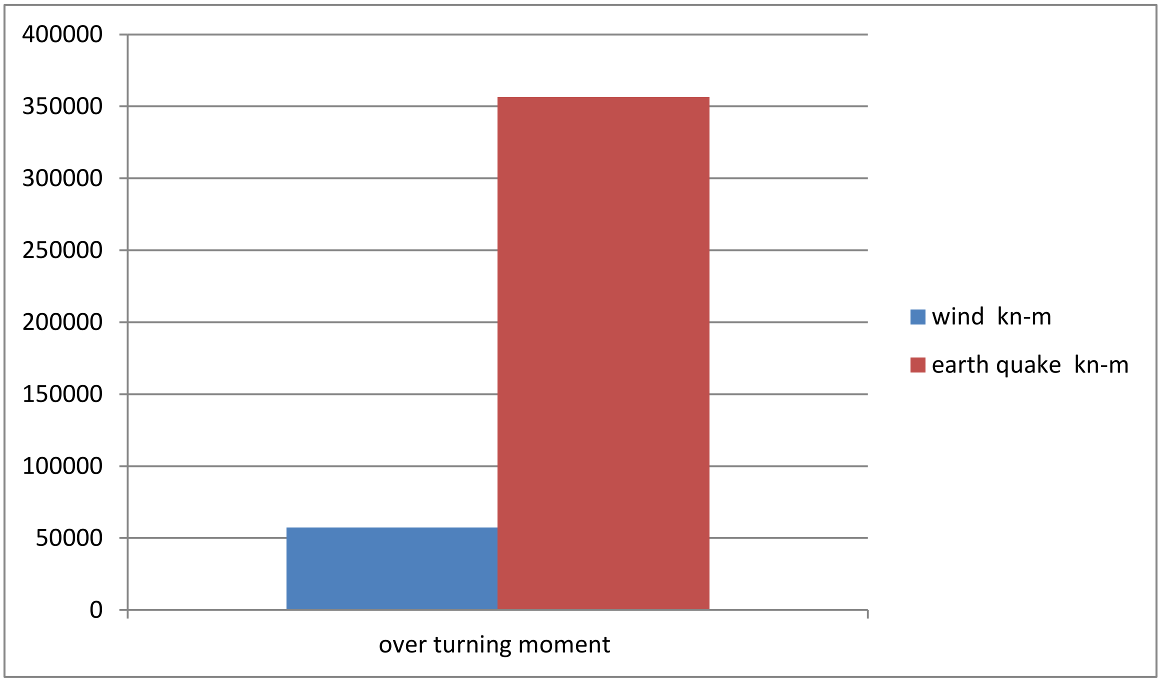


Fig.7.2 overturning moment comparison for wind and earthquake.

Chapter 8
Modeling

8.0 Introduction

The modeling water tanker is made in SAP2000 V18 Software. In the modeling all shell elements are modeled as a thin shell element with appropriate stiffness modifier. For the beams and columns stiffness modifier according to ACI code have been used.

The loading is defined in load Pattern definition dialog box of SAP2000. For the definition of wind load, EURO code-1 2004 has been used with appropriate side coefficients. Earthquake load is defined using response spectrum load case. After defining all load cases and patterns, the loads are combined following the EBCS (Ethiopian Building Codes of Standard) rules of combination.

Definition of material properties, frame sections and areal elements has been performed. Then the modeling (drawing of each element) is done. Then load is applied to the corresponding elements.

Analysis and design of the shells, beams and columns are also performed in SAP2000 program.

8.1 Material definition

Two materials are used in water tanker design. Concrete with a grade of 25MPa and Reinforcement of grade S-400.

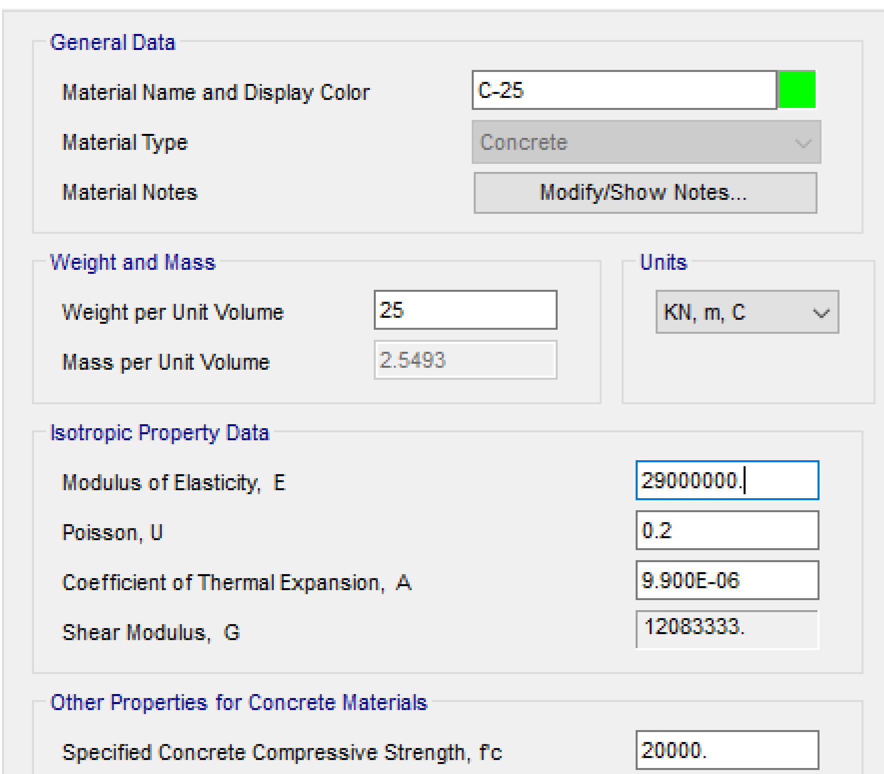


Fig.8.1.Concerte material definition

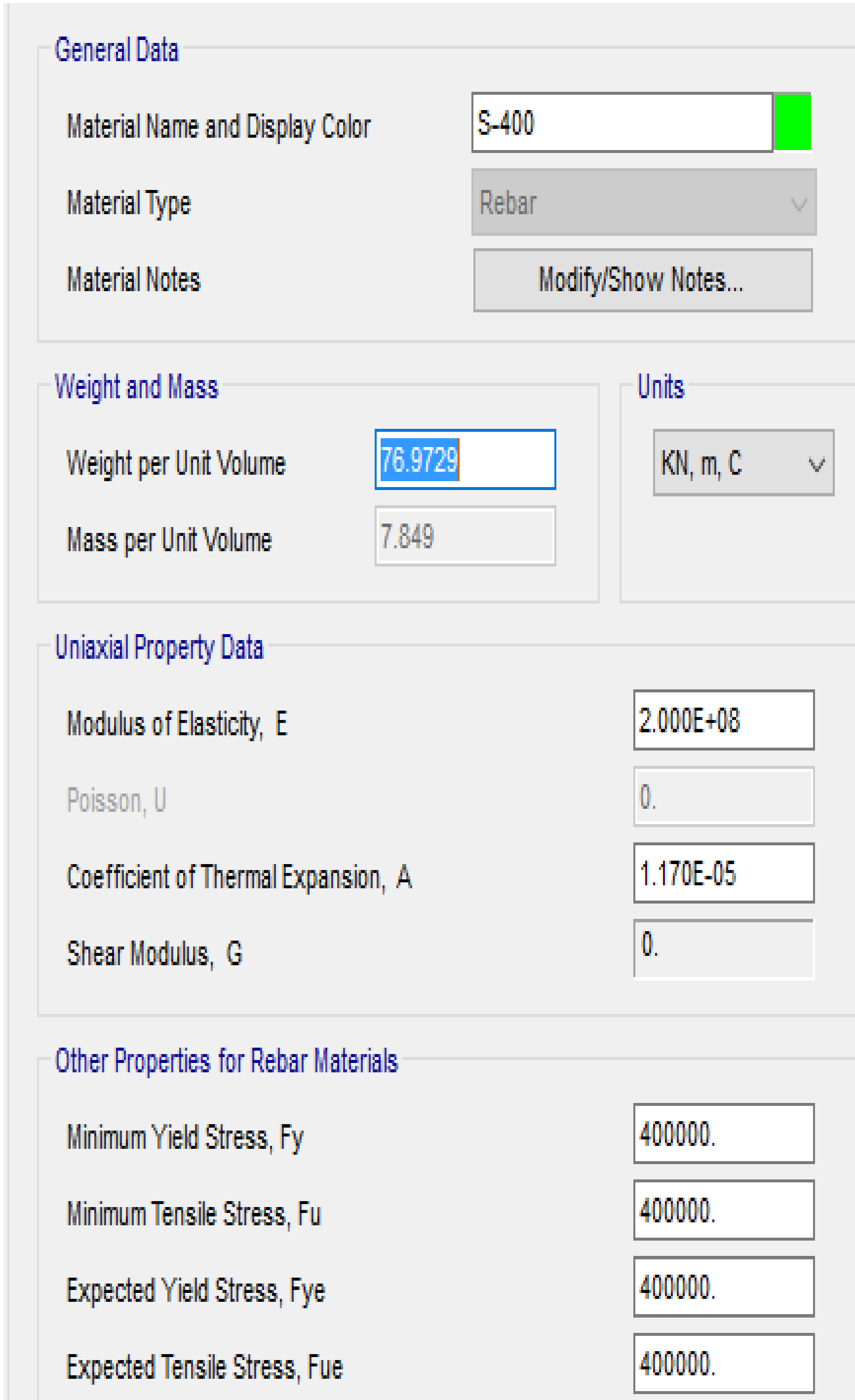


Fig.8.2. Reinforcement bar definition

For the definition of reinforcement bar property rigid plastic stress vs strain is assumed which is curve B in the figure below.

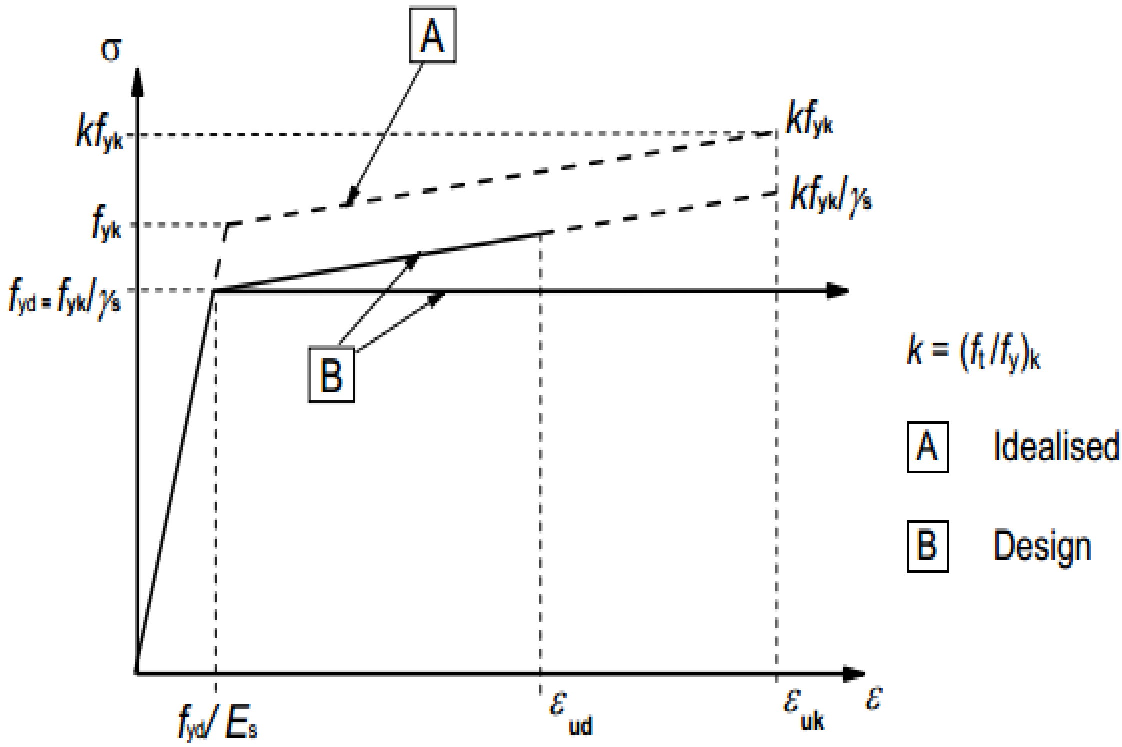
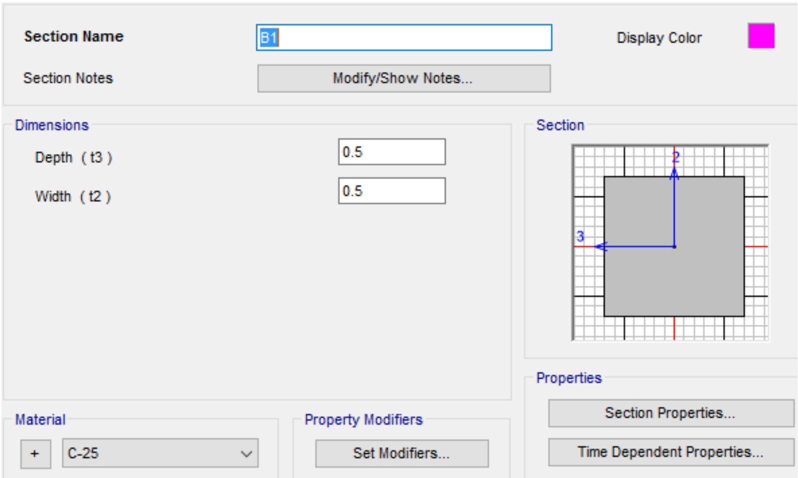


Fig.8.3.Idealized rebar stress-strain profile

8.2 Frame section definition

There are three beams in the water tanker. The first one is the top ring beam located at intersection of top dome and the cylinder which have a dimension of 500mm by 500mm, in SAP2000 it is labeled as B1.The second one is the circular ring beam located at intersection of the cylinder and conical dome which have a dimension of 1200mm by 700mm, in SAP2000 it is labeled as B2.The third one is the circular ring beam located at intersection of the bottom dome and columns which have a dimension of 1200mm by 700mm, in SAP2000 it is labeled as B3. All twelve columns are circular and have a dimension of diameter 800mm.



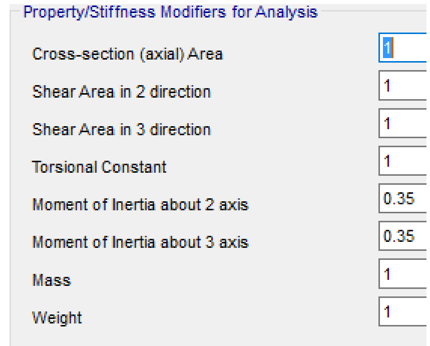
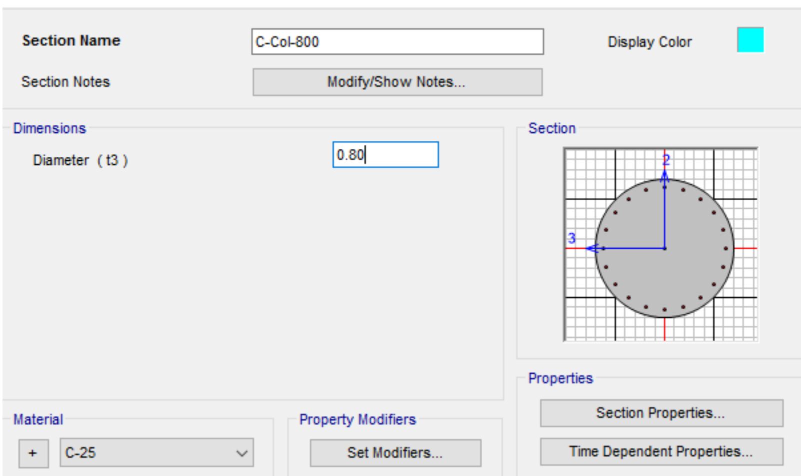


Fig.8.5. Stiffness modifier

Fig.8.4.Beam definition



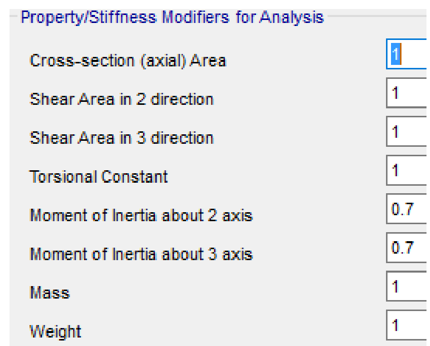


Fig.8.7. Stiffness modifier

Fig.8.6. Column definition

The other two beams (B-2 and B-3) are also defined with similar fashion as in B-1 definition.

8.3 Area element definition

A total of four shells are used in the water tanker modeling and analysis. This is top dome used as roof system, cylindrical wall, conical dome and bottom dome. The top dome has a diameter of 12m and thickness of 130mm. The cylindrical has a diameter of 12m with thickness varying from 200mm at the top to 450mm at the bottom where the height of the cylinder is 9m. The conical dome has a thickness of 650 mm. The bottom dome has a diameter of 7.5m and thickness of 330mm.

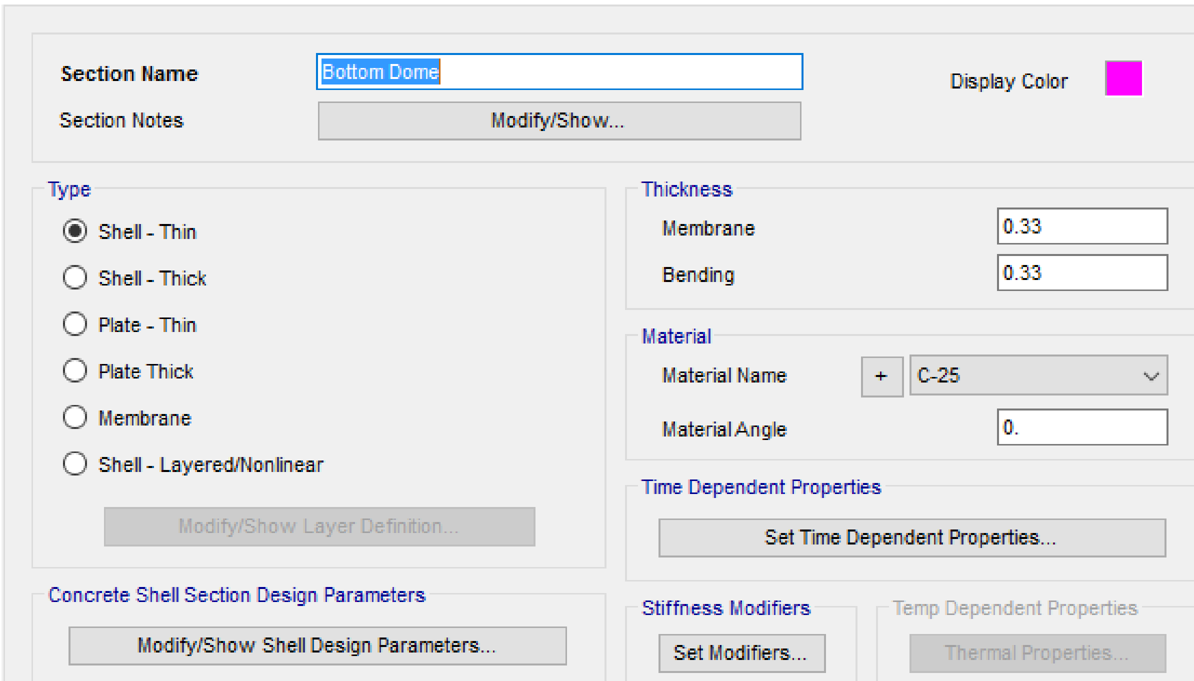


Fig.8.8. Area element definition in SAP2000

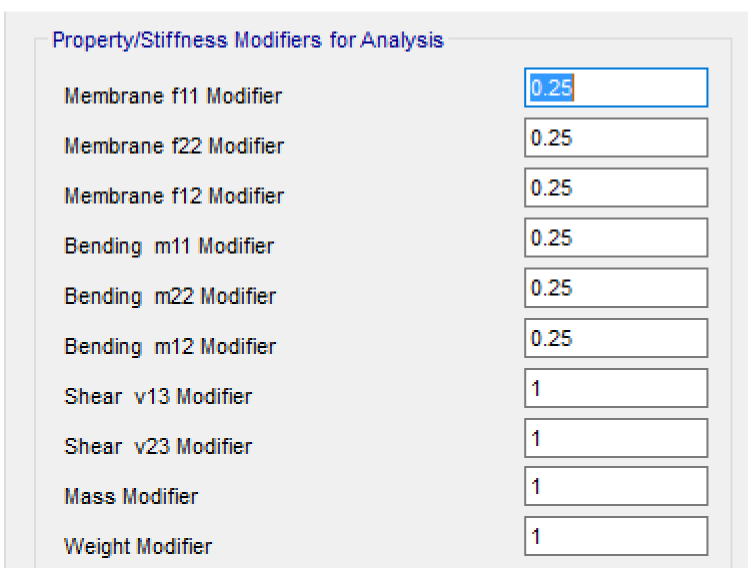


Fig.8.9. Stiffness modifiers

8.4 Load and combination definition

A total of nine load patterns are defined and there are ten load cases including response spectrum for earthquake. The total number of load combination is eight.

TABLE: Load Pattern Definitions		
Load Pattern	**Design Type**	**Self-Weight Multiplier**
Text	Text	Unit less
Self-Weight	DEAD	1
Super Dead	SUPER DEAD	0
Water Full	SUPER DEAD	0
Water Half	SUPER DEAD	0
Lateral Pressure Full	SUPER DEAD	0
Lateral Pressure Half	SUPER DEAD	0
Sloshing	SUPER DEAD	0
Live Roof	ROOF LIVE	0
WIND	WIND	0

Table 8.1. Load pattern definition

TABLE: Load Case Definitions				
Case	**Type**	**Design Type**	**Des Act Opt**	**Design Action**
Text	Text	Text	Text	Text
Self-Weight	LinStatic	DEAD	ProgDet	Non-Composite
Super Dead	LinStatic	SUPER DEAD	ProgDet	Long-Term Composite
Water Full	LinStatic	SUPER DEAD	ProgDet	Long-Term Composite
Water Half	LinStatic	SUPER DEAD	ProgDet	Long-Term Composite
Lateral Pressure Full	LinStatic	SUPER DEAD	ProgDet	Long-Term Composite
Lateral Pressure Half	LinStatic	SUPER DEAD	ProgDet	Long-Term Composite
Sloshing	LinStatic	SUPER DEAD	ProgDet	Long-Term Composite
Live Roof	LinStatic	ROOF LIVE	ProgDet	Short-Term Composite
WIND	LinStatic	WIND	ProgDet	Short-Term Composite
MODAL	LinModal	OTHER	ProgDet	Other
RSA	LinRespSpec	QUAKE	ProgDet	Short-Term Composite

Table 8.2. Load case definition

Super dead load accounts to the additional load coming from cement screed and plastering and other dead loads.

Response spectrum is defined based on Eurocode-1998.

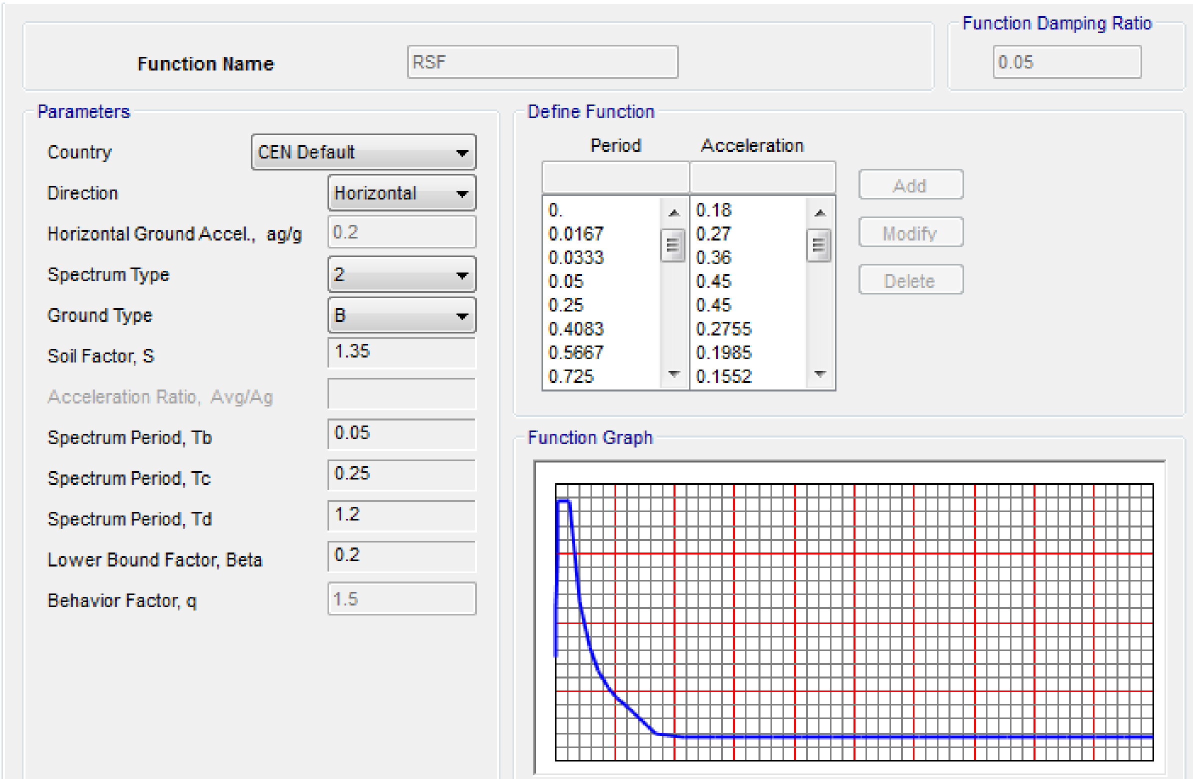


Fig.8.10 Response spectrum function definition

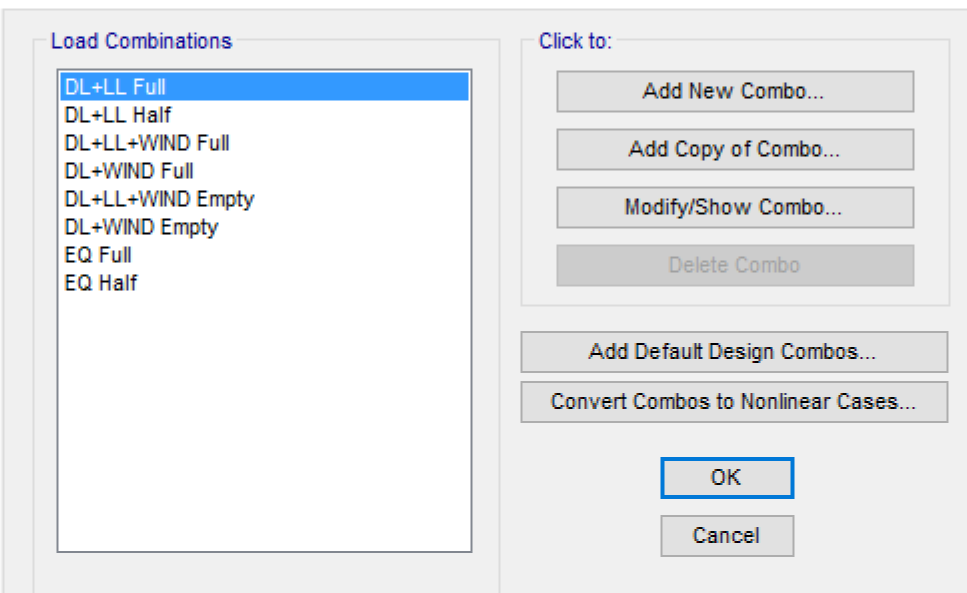


Fig.8.11 Load combinations

The sloshing effect is considered when the water in the tanker is half. Sloshing effect has positive effect on the performance of the structure during earthquake. But to see whether this is correct or not it has been included in the analysis.

For the wind load combination when the water in the tanker is empty will be the governing combination but for completeness when the water is full wind load is applied.

8.5 Modeling

The step by step modeling of the water tanker follows the following procedures:

- First columns are modeled then the beams (bracing beams) are drawn.
- Then bottom are modeled
- Ring beam three is modeled then the cylinder shell is modeled
- Top ring mean is modeled
- Finally, the top dome is modeled

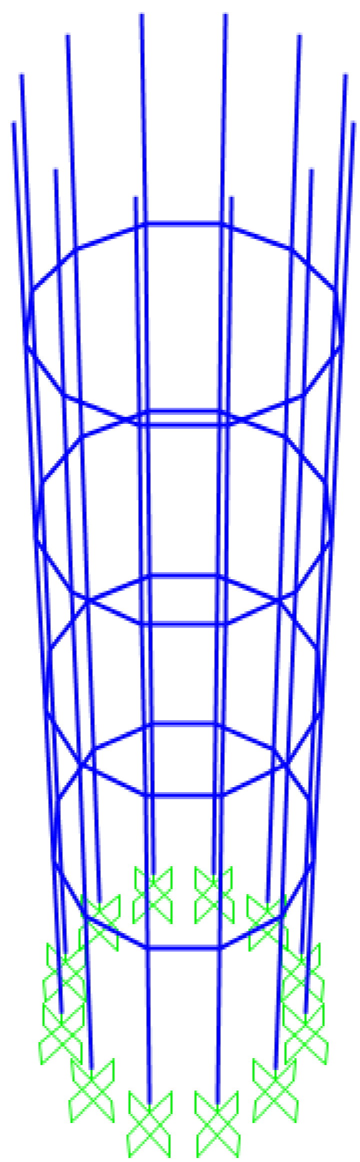


Fig.8.12. Column and bracing beam model

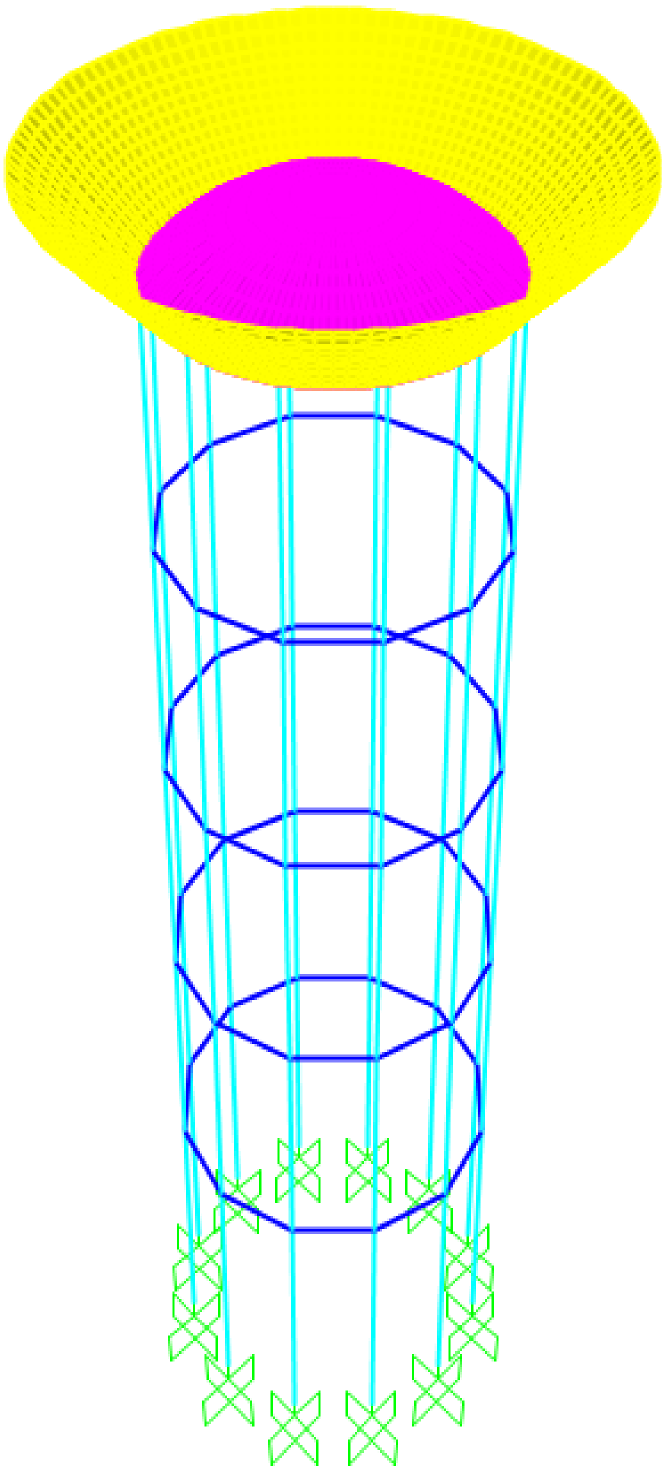


Fig.8.13.Conical dome

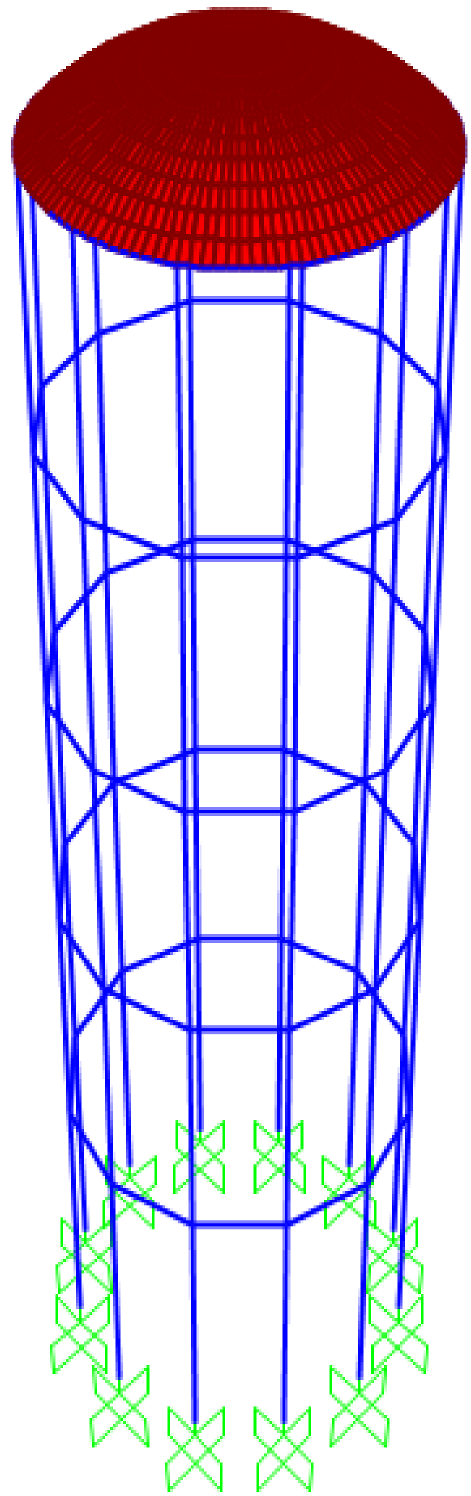


Fig.8.14.Bottom dome

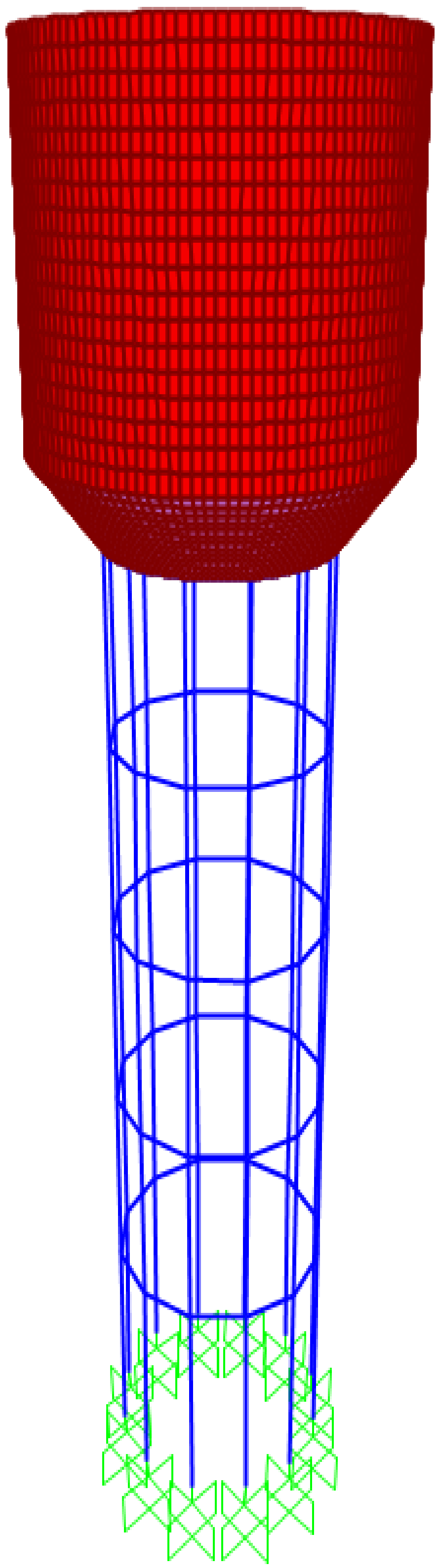


Fig.8.15. Cylindrical shell

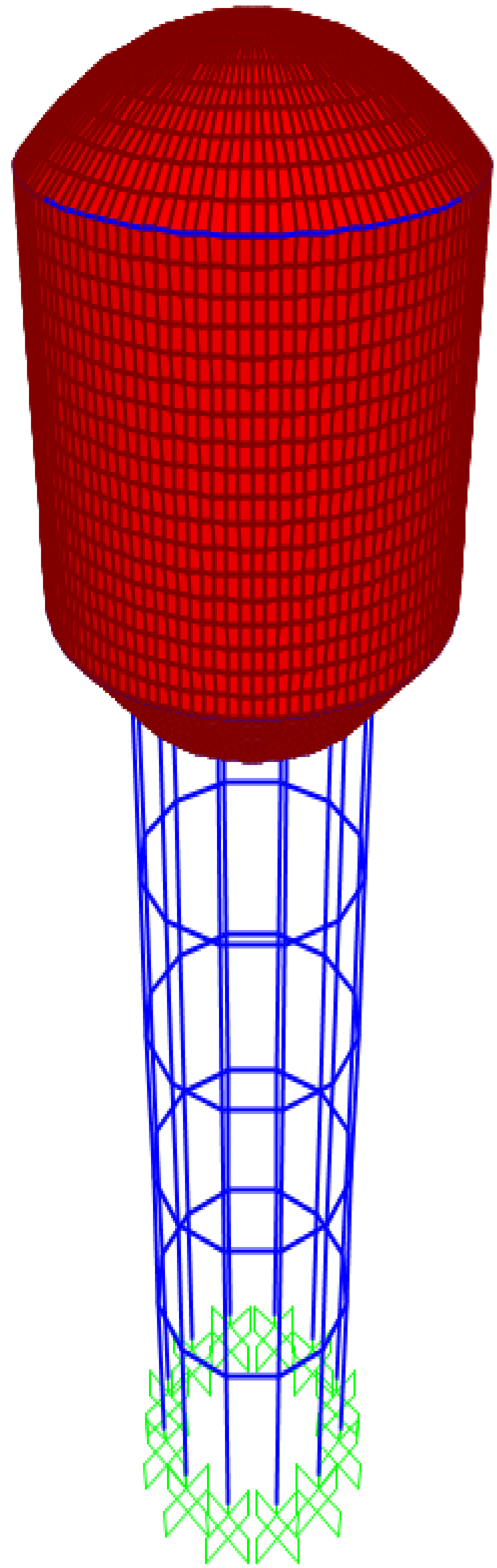


Fig.8.16. Top Dome

Chapter 9
Analysis Result

9.0 Introduction

For the analysis of elevated water tanker linear elastic analysis method has been used. After all loadings are applied to the corresponding elements analysis is carried out. First standard solver is used to check whether a warning or error message has been generated. The analysis was full scale three-dimensional analysis which has all six degree of freedom.

```
SAP2000 v18.1.1 Ultimate 64-bit (Analysis Build 9447/64)
 File: D:\Projects\Water Tanker\SAP2000 Model\model-1-15.LOG



 B E G I N   A N A L Y S I S
2016/07/13  17:14:30


 RUNNING ANALYSIS AS A SEPARATE PROCESS
 USING THE STANDARD SOLVER (PROVIDES COMPLETE INSTABILITY INFORMATION)

 NUMBER OF JOINTS                             =       5707
    WITH RESTRAINTS                           =         12
 NUMBER OF FRAME/CABLE/TENDON ELEMENTS        =        528
 NUMBER OF SHELL ELEMENTS                     =       5760
 NUMBER OF CONSTRAINTS/WELDS                  =         49
 NUMBER OF LOAD PATTERNS                      =          9
 NUMBER OF ACCELERATION LOADS                 =          9
 NUMBER OF LOAD CASES                         =         11


 E L E M E N T   F O R M A T I O N
17:14:30



 L I N E A R   E Q U A T I O N   S O L U T I O N
17:14:31

 FORMING STIFFNESS AT ZERO (UNSTRESSED) INITIAL CONDITIONS

 TOTAL NUMBER OF EQUILIBRIUM EQUATIONS        =      34170
 APPROXIMATE "EFFECTIVE" BAND WIDTH           =        695

 NUMBER OF EQUATION STORAGE BLOCKS            =          1
 MAXIMUM BLOCK SIZE (8-BYTE TERMS)            =   23522799
 SIZE OF STIFFNESS FILE(S)                    =    179.595 MB
```

```
 NUMBER OF EQUATIONS TO SOLVE              =       34170

 ---------------------------------
 BASIC STABILITY CHECK FOR LINEAR LOAD CASES:
    NUMBER OF NEGATIVE STIFFNESS EIGENVALUES SHOULD BE ZERO FOR
STABILITY.
    (NOTE: FURTHER CHECKS SHOULD BE CONSIDERED AS DEEMED NECESSARY,
     SUCH AS REVIEWING EIGEN MODES FOR MECHANISMS AND RIGID-BODY
MOTION)

    NUMBER OF NEGATIVE EIGENVALUES         =           0, OK.
 ---------------------------------


 L I N E A R   S T A T I C   C A S E S
17:14:46

 USING STIFFNESS AT ZERO (UNSTRESSED) INITIAL CONDITIONS

 TOTAL NUMBER OF CASES TO SOLVE            =           9
 NUMBER OF CASES TO SOLVE PER BLOCK        =           9

 LINEAR STATIC CASES TO BE SOLVED:

 CASE: SELF WEIGHT
 CASE: SUPER DEAD
 CASE: WATET FULL
 CASE: WATET HALF
 CASE: LATERAL PRESSURE FULL
 CASE: LATERAL PRESSURE HALF
 CASE: SLOSHING
 CASE: LIVE ROOF
 CASE: WIND


 E I G E N   M O D A L   A N A L Y S I S
17:14:46

 CASE: MODAL

 USING STIFFNESS AT ZERO (UNSTRESSED) INITIAL CONDITIONS

 NUMBER OF STIFFNESS DEGREES OF FREEDOM    =       34170
 NUMBER OF MASS DEGREES OF FREEDOM         =       17085
 MAXIMUM NUMBER OF EIGEN MODES SOUGHT      =          12
 MINIMUM NUMBER OF EIGEN MODES SOUGHT      =           1
 NUMBER OF RESIDUAL-MASS MODES SOUGHT      =           0
 NUMBER OF SUBSPACE VECTORS USED           =          24
```

```
 RELATIVE CONVERGENCE TOLERANCE            =    1.00E-09

 FREQUENCY SHIFT (CENTER) (CYC/TIME)      =     .000000
 FREQUENCY CUTOFF (RADIUS) (CYC/TIME)      = -INFINITY-
 ALLOW AUTOMATIC FREQUENCY SHIFTING        =         YES
 Original stiffness at shift: EV= 0.0000000E+00, f=     .000000, T= -
INFINITY-
 Number of eigenvalues below shift =      0
 Found mode      1 of     12:  EV= 7.1051740E+00, f=    0.424236, T=
2.357179
 Found mode      2 of     12:  EV= 7.1051740E+00, f=    0.424236, T=
2.357179
 Found mode      3 of     12:  EV= 1.1232370E+01, f=    0.533403, T=
1.874753
 Found mode      4 of     12:  EV= 3.8470156E+02, f=    3.121635, T=
0.320345
 Found mode      5 of     12:  EV= 3.8470156E+02, f=    3.121635, T=
0.320345
 Found mode      6 of     12:  EV= 7.5500208E+02, f=    4.373148, T=
0.228668
 Found mode      7 of     12:  EV= 7.5500208E+02, f=    4.373148, T=
0.228668
 Found mode      8 of     12:  EV= 9.3620964E+02, f=    4.869750, T=
0.205349
 Found mode      9 of     12:  EV= 2.3452821E+03, f=    7.707574, T=
0.129743
 Found mode     10 of     12:  EV= 2.3644308E+03, f=    7.738975, T=
0.129216
 Found mode     11 of     12:  EV= 2.3644308E+03, f=    7.738975, T=
0.129216
 Found mode     12 of     12:  EV= 3.8383363E+03, f=    9.860334, T=
0.101416

 NUMBER OF EIGEN MODES FOUND               =          12
 NUMBER OF ITERATIONS PERFORMED            =          11
 NUMBER OF STIFFNESS SHIFTS                =           0

 R E S P O N S E - S P E C T R U M   A N A L Y S I S
17:14:53
 CASE: RSA
 TYPE OF EXCITATION                    = STANDARD GROUND
ACCELERATION

 USING MODES FROM CASE: MODAL
 NUMBER OF DYNAMIC MODES TO BE USED        =          12


 A N A L Y S I S   C O M P L E T E
2016/07/13  17:14:53
```

As can been seen from the above text taken from SAP2000 the is no warning or error messagereported. After this verification standard solver is changed to advanced solver.

9.1 Labeling of elements

The columns are divided in to five parts along its height, since columns with height above five to eight (5-8) meters generally will be very slender depending on the size of beam and column as well as the supporting condition. For this elevated water tanker, the bottom column height is 5.75m and the rest columns are 5m in length. This is made possible by providing diagonal bracing of the columns. All columns are reinforced concrete frames having a circular shape and diameter of 800mm.

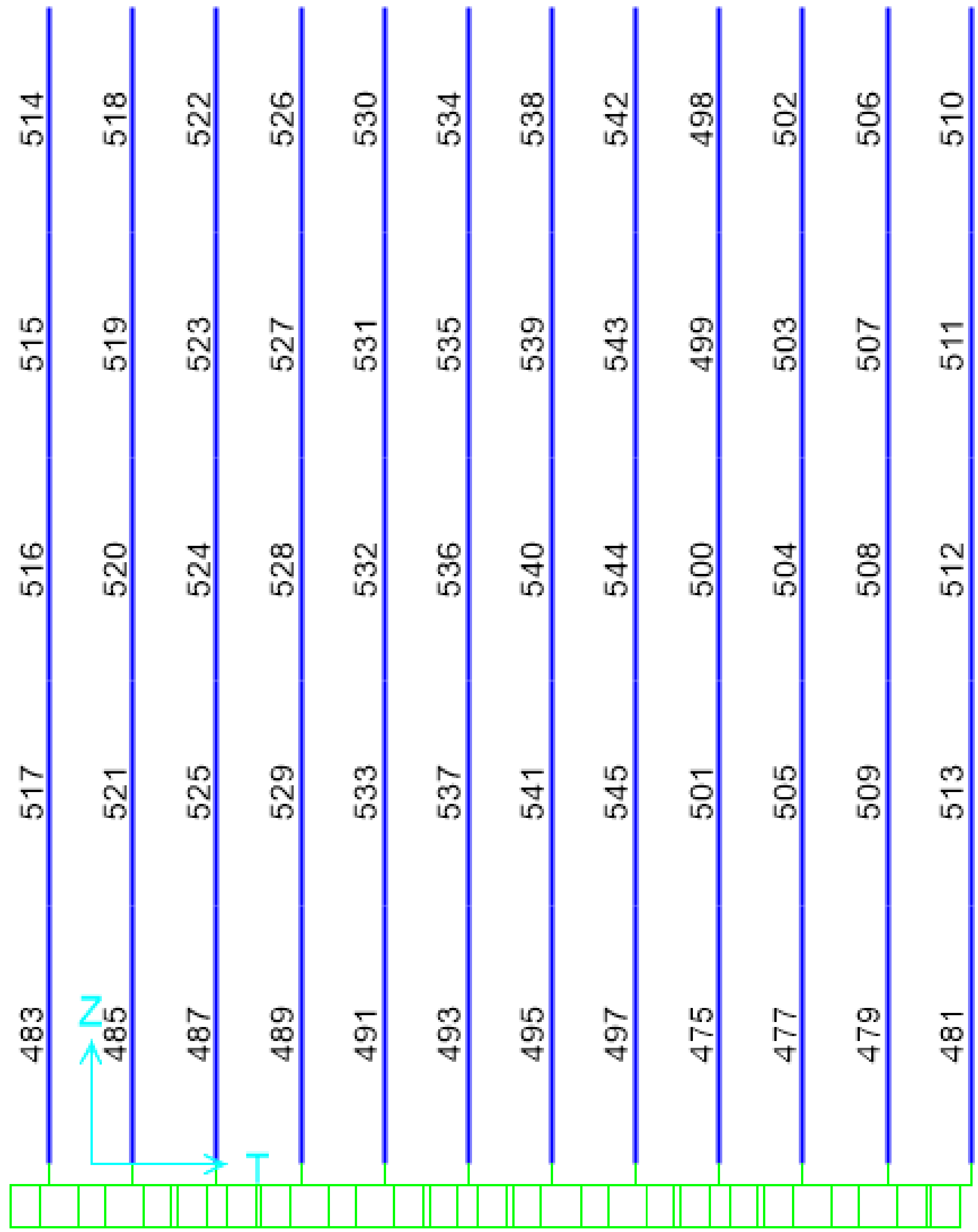


Fig.9.1. Column label

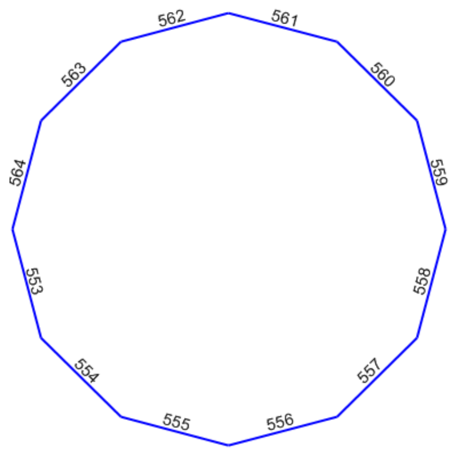


Fig.9.2. Bracing beam @5.75m

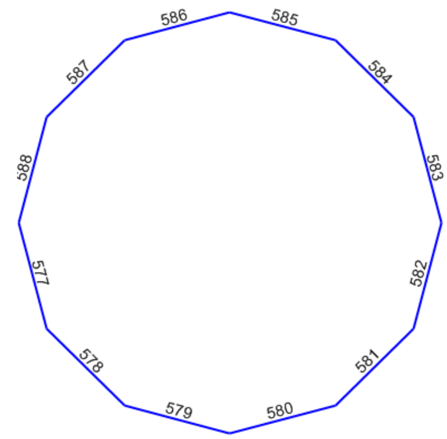


Fig.9.3. Bracing beam @10.75m

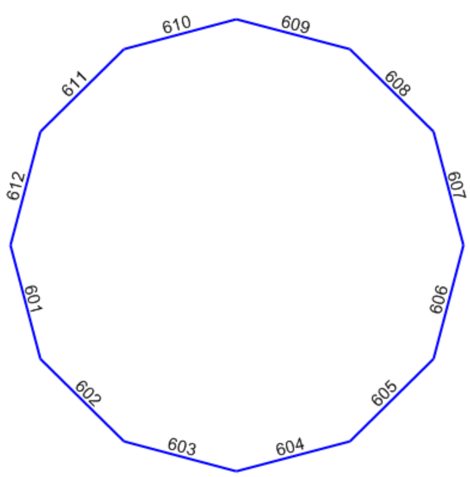


Fig.9.4.Bracing beam @15.75m

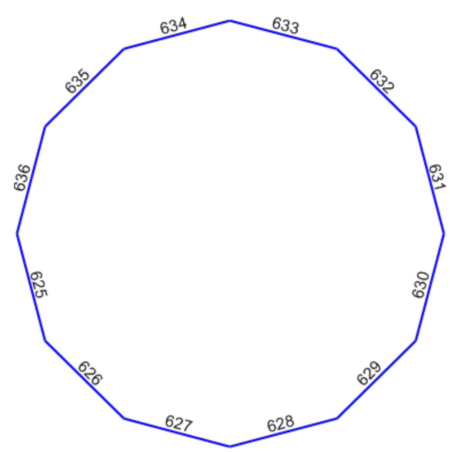


Fig.9.5. Bracing beam @20.75m

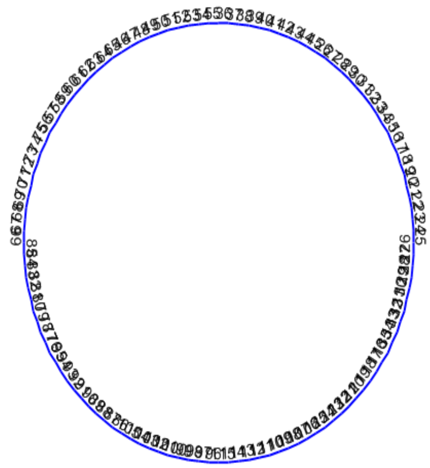


Fig.9.6.Bracing beam @25.75m

9.2 Results

9.2.1 Beam and Column forces

TABLE: Element Forces - Frames									
Frame	Station	Output Case	Case Type	Step Type	P	V2	V3	M2	M3
Text	m	Text	Text	Text	KN	KN	KN	KN-m	KN-m
1	0	DL+LL Full	Combination		-239.224	-30.469	-2.318E-08	-7.189E-08	-43.031
1	0.46875	DL+LL Full	Combination		-239.224	-26.66	-2.318E-08	-6.102E-08	-29.6414
1	0.9375	DL+LL Full	Combination		-239.224	-22.852	-2.318E-08	-5.016E-08	-18.0371
1	1.40625	DL+LL Full	Combination		-239.224	-19.043	-2.318E-08	-3.929E-08	-8.2181
1	1.875	DL+LL Full	Combination		-239.224	-15.234	-2.318E-08	-2.843E-08	-0.1843
1	2.34375	DL+LL Full	Combination		-239.224	-11.426	-2.318E-08	-1.756E-08	6.0642
1	2.8125	DL+LL Full	Combination		-239.224	-7.617	-2.318E-08	-6.694E-09	10.5274
1	3.28125	DL+LL Full	Combination		-239.224	-3.809	-2.318E-08	4.171E-09	13.2053
1	3.75	DL+LL Full	Combination		-239.224	-5.24E-08	-2.318E-08	1.504E-08	14.0979
1	0	DL+LL Half	Combination		-227.35	-30.469	-1.429E-08	-4.671E-08	-41.927
1	0.46875	DL+LL Half	Combination		-227.35	-26.66	-1.429E-08	-4.001E-08	-28.5374
1	0.9375	DL+LL Half	Combination		-227.35	-22.852	-1.429E-08	-3.332E-08	-16.9331
1	1.40625	DL+LL Half	Combination		-227.35	-19.043	-1.429E-08	-2.662E-08	-7.1141
1	1.875	DL+LL Half	Combination		-227.35	-15.234	-1.429E-08	-1.992E-08	0.9196
1	2.34375	DL+LL Half	Combination		-227.35	-11.426	-1.429E-08	-1.323E-08	7.1681
1	2.8125	DL+LL Half	Combination		-227.35	-7.617	-1.429E-08	-6.53E-09	11.6313
1	3.28125	DL+LL Half	Combination		-227.35	-3.809	-1.429E-08	1.671E-10	14.3092
1	3.75	DL+LL Half	Combination		-227.35	-7.574E-08	-1.429E-08	6.864E-09	15.2019
1	0	DL+LL+WIND Full	Combination		-305.796	-22.674	0.046	0.1162	-20.1071
1	0.46875	DL+LL+WIND Full	Combination		-305.796	-18.866	0.046	0.0948	-10.3711
1	0.9375	DL+LL+WIND Full	Combination		-305.796	-15.057	0.046	0.0734	-2.4205
1	1.40625	DL+LL+WIND Full	Combination		-305.796	-11.249	0.046	0.0521	3.7449
1	1.875	DL+LL+WIND Full	Combination		-305.796	-7.44	0.046	0.0307	8.125
1	2.34375	DL+LL+WIND Full	Combination		-305.796	-3.631	0.046	0.0093	10.7199
1	2.8125	DL+LL+WIND Full	Combination		-305.796	0.177	0.046	-0.0121	11.5294
1	3.28125	DL+LL+WIND Full	Combination		-305.796	3.986	0.046	-0.0334	10.5537
1	3.75	DL+LL+WIND Full	Combination		-305.796	7.794	0.046	-0.0548	7.7927
1	0	DL+WIND Full	Combination		-316.284	-21.231	0.054	0.1377	-15.8401
1	0.46875	DL+WIND Full	Combination		-316.284	-17.422	0.054	0.1124	-6.7807
1	0.9375	DL+WIND Full	Combination		-316.284	-13.614	0.054	0.087	0.4933
1	1.40625	DL+WIND Full	Combination		-316.284	-9.805	0.054	0.0617	5.9821
1	1.875	DL+WIND Full	Combination		-316.284	-5.997	0.054	0.0364	9.6856
1	2.34375	DL+WIND Full	Combination		-316.284	-2.188	0.054	0.011	11.6039
1	2.8125	DL+WIND Full	Combination		-316.284	1.621	0.054	-0.0143	11.7368
1	3.28125	DL+WIND Full	Combination		-316.284	5.429	0.054	-0.0396	10.0845

1	3.75	DL+WIND Full	Combination		-316.284	9.238	0.054	-0.065	6.6469
1	0	DL+LL+WIND Empty	Combination		-283.48	-22.674	0.046	0.1162	-17.8414
1	0.46875	DL+LL+WIND Empty	Combination		-283.48	-18.866	0.046	0.0948	-8.1054
1	0.9375	DL+LL+WIND Empty	Combination		-283.48	-15.057	0.046	0.0734	-0.1548
1	1.40625	DL+LL+WIND Empty	Combination		-283.48	-11.249	0.046	0.0521	6.0106
1	1.875	DL+LL+WIND Empty	Combination		-283.48	-7.44	0.046	0.0307	10.3907
1	2.34375	DL+LL+WIND Empty	Combination		-283.48	-3.631	0.046	0.0093	12.9856
1	2.8125	DL+LL+WIND Empty	Combination		-283.48	0.177	0.046	-0.0121	13.7951
1	3.28125	DL+LL+WIND Empty	Combination		-283.48	3.986	0.046	-0.0334	12.8194
1	3.75	DL+LL+WIND Empty	Combination		-283.48	7.794	0.046	-0.0548	10.0584
1	0	DL+WIND Empty	Combination		-293.968	-21.231	0.054	0.1377	-13.5744
1	0.46875	DL+WIND Empty	Combination		-293.968	-17.422	0.054	0.1124	-4.515
1	0.9375	DL+WIND Empty	Combination		-293.968	-13.614	0.054	0.087	2.759
1	1.40625	DL+WIND Empty	Combination		-293.968	-9.805	0.054	0.0617	8.2478
1	1.875	DL+WIND Empty	Combination		-293.968	-5.997	0.054	0.0364	11.9513
1	2.34375	DL+WIND Empty	Combination		-293.968	-2.188	0.054	0.011	13.8696
1	2.8125	DL+WIND Empty	Combination		-293.968	1.621	0.054	-0.0143	14.0025
1	3.28125	DL+WIND Empty	Combination		-293.968	5.429	0.054	-0.0396	12.3502
1	3.75	DL+WIND Empty	Combination		-293.968	9.238	0.054	-0.065	8.9126
1	0	EQ Full	Combination	Max	-178.577	-18.078	0.144	0.101	-18.1746
1	0.46875	EQ Full	Combination	Max	-178.577	-15.222	0.144	0.0414	-10.3699
1	0.9375	EQ Full	Combination	Max	-178.577	-12.365	0.144	0.048	-3.9042
1	1.40625	EQ Full	Combination	Max	-178.577	-9.509	0.144	0.1094	1.2226
1	1.875	EQ Full	Combination	Max	-178.577	-6.652	0.144	0.1753	5.0105
1	2.34375	EQ Full	Combination	Max	-178.577	-3.796	0.144	0.2419	7.4597
1	2.8125	EQ Full	Combination	Max	-178.577	-0.94	0.144	0.3089	8.5717
1	3.28125	EQ Full	Combination	Max	-178.577	1.917	0.144	0.376	11.4694
1	3.75	EQ Full	Combination	Max	-178.577	4.773	0.144	0.4432	14.3756
1	0	EQ Full	Combination	Min	-180.259	-27.625	-0.144	-0.101	-46.3719
1	0.46875	EQ Full	Combination	Min	-180.259	-24.769	-0.144	-0.0414	-34.0922
1	0.9375	EQ Full	Combination	Min	-180.259	-21.912	-0.144	-0.048	-23.1515
1	1.40625	EQ Full	Combination	Min	-180.259	-19.056	-0.144	-0.1094	-13.5497
1	1.875	EQ Full	Combination	Min	-180.259	-16.199	-0.144	-0.1753	-5.287
1	2.34375	EQ Full	Combination	Min	-180.259	-13.343	-0.144	-0.2419	1.6366

1	2.8125	EQ Full	Combination	Min	-180.259	-10.486	-0.144	-0.3089	7.2193
1	3.28125	EQ Full	Combination	Min	-180.259	-7.63	-0.144	-0.376	8.3386
1	3.75	EQ Full	Combination	Min	-180.259	-4.773	-0.144	-0.4432	6.7712
1	0	EQ Half	Combination	Max	-171.54	-33.552	0.144	0.101	-63.2285
1	0.46875	EQ Half	Combination	Max	-171.54	-30.695	0.144	0.0414	-48.1706
1	0.9375	EQ Half	Combination	Max	-171.54	-27.839	0.144	0.048	-34.4517
1	1.40625	EQ Half	Combination	Max	-171.54	-24.982	0.144	0.1094	-22.0717
1	1.875	EQ Half	Combination	Max	-171.54	-22.126	0.144	0.1753	-11.0306
1	2.34375	EQ Half	Combination	Max	-171.54	-19.269	0.144	0.2419	-1.3282
1	2.8125	EQ Half	Combination	Max	-171.54	-16.413	0.144	0.3089	7.037
1	3.28125	EQ Half	Combination	Max	-171.54	-13.557	0.144	0.376	17.1879
1	3.75	EQ Half	Combination	Max	-171.54	-10.7	0.144	0.4432	27.3473
1	0	EQ Half	Combination	Min	-173.223	-43.098	-0.144	-0.101	-91.4258
1	0.46875	EQ Half	Combination	Min	-173.223	-40.242	-0.144	-0.0414	-71.8929
1	0.9375	EQ Half	Combination	Min	-173.223	-37.386	-0.144	-0.048	-53.699
1	1.40625	EQ Half	Combination	Min	-173.223	-34.529	-0.144	-0.1094	-36.844
1	1.875	EQ Half	Combination	Min	-173.223	-31.673	-0.144	-0.1753	-21.3281
1	2.34375	EQ Half	Combination	Min	-173.223	-28.816	-0.144	-0.2419	-7.1513
1	2.8125	EQ Half	Combination	Min	-173.223	-25.96	-0.144	-0.3089	5.6846
1	3.28125	EQ Half	Combination	Min	-173.223	-23.103	-0.144	-0.376	14.0571
1	3.75	EQ Half	Combination	Min	-173.223	-20.247	-0.144	-0.4432	19.7429

Table 9.1. Bracing beam forces for label-1

TABLE: Element Forces - Frames									
Frame	**Station**	**Output Case**	**Case Type**	**StepType**	**P**	**V2**	**V3**	**M2**	**M3**
Text	m	Text	Text	Text	KN	KN	KN	KN-m	KN-m
475	2.875	DL+LL Full	Combination		-2883.95	5.137E-11	-3.869	-3.765	3.992E-08
475	5.75	DL+LL Full	Combination		-2930.916	5.137E-11	-3.869	7.3594	3.977E-08
475	0	DL+LL Half	Combination		-2236.847	-8.517E-10	-3.887	-14.9581	1.973E-08
475	2.875	DL+LL Half	Combination		-2283.814	-8.517E-10	-3.887	-3.7831	2.218E-08
475	5.75	DL+LL Half	Combination		-2330.781	-8.517E-10	-3.887	7.3919	2.463E-08
475	0	DL+LL+WIND Full	Combination		-2836.94	-111.803	-3.867	-14.8805	-184.8246
475	2.875	DL+LL+WIND Full	Combination		-2883.907	-111.803	-3.867	-3.7628	136.6104
475	5.75	DL+LL+WIND Full	Combination		-2930.873	-111.803	-3.867	7.3548	458.0453

475	0	DL+WIND Full	Combination		-2829.829	-132.508	-3.867	-14.8806	-219.0514
475	2.875	DL+WIND Full	Combination		-2876.795	-132.508	-3.867	-3.7629	161.9086
475	5.75	DL+WIND Full	Combination		-2923.762	-132.508	-3.867	7.3548	542.8685
475	0	DL+LL+WIND Empty	Combination		-1636.669	-111.803	-3.903	-15.0209	-184.8246
475	2.875	DL+LL+WIND Empty	Combination		-1683.636	-111.803	-3.903	-3.7998	136.6104
475	5.75	DL+LL+WIND Empty	Combination		-1730.603	-111.803	-3.903	7.4213	458.0453
475	0	DL+WIND Empty	Combination		-1629.558	-132.508	-3.903	-15.021	-219.0514
475	2.875	DL+WIND Empty	Combination		-1676.524	-132.508	-3.903	-3.7999	161.9086
475	5.75	DL+WIND Empty	Combination		-1723.491	-132.508	-3.903	7.4213	542.8685
475	0	EQ Full	Combination	Max	-1514.657	72.853	36.692	9.3921	123.955
475	2.875	EQ Full	Combination	Max	-1549.882	72.853	36.692	91.0452	86.2598
475	5.75	EQ Full	Combination	Max	-1585.107	72.853	36.692	213.1645	295.3937
475	0	EQ Full	Combination	Min	-2740.817	-72.853	-42.496	-31.7262	-123.955
475	2.875	EQ Full	Combination	Min	-2776.042	-72.853	-42.496	-96.6927	-86.2598
475	5.75	EQ Full	Combination	Min	-2811.267	-72.853	-42.496	-202.1254	-295.3937
475	0	EQ Half	Combination	Max	-1064.555	264.352	36.679	9.3409	437.6662
475	2.875	EQ Half	Combination	Max	-1099.78	264.352	36.679	91.0317	-150.5881
475	5.75	EQ Half	Combination	Max	-1135.006	264.352	36.679	213.1887	-492.0134
475	0	EQ Half	Combination	Min	-2290.716	118.646	-42.51	-31.7774	189.7563
475	2.875	EQ Half	Combination	Min	-2325.941	118.646	-42.51	-96.7062	-323.1077
475	5.75	EQ Half	Combination	Min	-2361.166	118.646	-42.51	-202.1012	-1082.8007
477	0	DL+LL Full	Combination		-2836.983	-1.935	-3.351	-12.8946	-7.4447
477	2.875	DL+LL Full	Combination		-2883.95	-1.935	-3.351	-3.2606	-1.8825
477	5.75	DL+LL Full	Combination		-2930.916	-1.935	-3.351	6.3734	3.6797
477	0	DL+LL Half	Combination		-2236.847	-1.943	-3.366	-12.9541	-7.479
477	2.875	DL+LL Half	Combination		-2283.814	-1.943	-3.366	-3.2763	-1.8915
477	5.75	DL+LL Half	Combination		-2330.781	-1.943	-3.366	6.4016	3.6959
477	0	DL+LL+WIND Full	Combination		-3430.569	-103.907	13.681	52.5146	-154.5131
477	2.875	DL+LL+WIND Full	Combination		-3477.535	-103.907	13.681	13.1829	144.2196
477	5.75	DL+LL+WIND Full	Combination		-3524.502	-103.907	13.681	-26.1488	442.9524
477	0	DL+WIND Full	Combination		-3533.389	-122.791	16.834	64.6259	-181.7489

477	2.875	DL+WIND Full	Combination		-3580.355	-122.791	16.834	16.2276	171.2753
477	5.75	DL+WIND Full	Combination		-3627.322	-122.791	16.834	-32.1707	524.2996
477	0	DL+LL+WIND Empty	Combination		-2230.298	-103.925	13.649	52.393	-154.5834
477	2.875	DL+LL+WIND Empty	Combination		-2277.264	-103.925	13.649	13.1509	144.2011
477	5.75	DL+LL+WIND Empty	Combination		-2324.231	-103.925	13.649	-26.0913	442.9856
477	0	DL+WIND Empty	Combination		-2333.118	-122.809	16.803	64.5043	-181.8192
477	2.875	DL+WIND Empty	Combination		-2380.085	-122.809	16.803	16.1956	171.2568
477	5.75	DL+WIND Empty	Combination		-2427.051	-122.809	16.803	-32.1132	524.3328
477	0	EQ Full	Combination	Max	-1600.231	68.753	47.157	57.4856	110.7736
477	2.875	EQ Full	Combination	Max	-1635.456	68.753	47.157	83.2407	88.2246
477	5.75	EQ Full	Combination	Max	-1670.681	68.753	47.157	230.5195	292.5997
477	0	EQ Full	Combination	Min	-2655.244	-71.655	-52.183	-76.8274	-121.9406
477	2.875	EQ Full	Combination	Min	-2690.469	-71.655	-52.183	-88.1316	-91.0484
477	5.75	EQ Full	Combination	Min	-2725.694	-71.655	-52.183	-220.9594	-287.0802
477	0	EQ Half	Combination	Max	60.973	243.412	17.964	-54.6288	359.8082
477	2.875	EQ Half	Combination	Max	25.748	243.412	17.964	55.055	-164.883
477	5.75	EQ Half	Combination	Max	-9.477	243.412	17.964	286.2625	-462.6502
477	0	EQ Half	Combination	Min	-994.04	103.003	-81.376	-188.9418	127.0941
477	2.875	EQ Half	Combination	Min	-1029.265	103.003	-81.376	-116.3173	-344.156
477	5.75	EQ Half	Combination	Min	-1064.49	103.003	-81.376	-165.2165	-1042.3302
479	0	DL+LL Full	Combination		-2836.983	-3.351	-1.935	-7.4447	-12.8946
479	2.875	DL+LL Full	Combination		-2883.95	-3.351	-1.935	-1.8825	-3.2606
479	5.75	DL+LL Full	Combination		-2930.916	-3.351	-1.935	3.6797	6.3734
479	0	DL+LL Half	Combination		-2236.847	-3.366	-1.943	-7.479	-12.9541
479	2.875	DL+LL Half	Combination		-2283.814	-3.366	-1.943	-1.8915	-3.2763
479	5.75	DL+LL Half	Combination		-2330.781	-3.366	-1.943	3.6959	6.4016
479	0	DL+LL+WIND Full	Combination		-3865.191	-85.658	15.096	57.9615	-84.4374
479	2.875	DL+LL+WIND Full	Combination		-3912.158	-85.658	15.096	14.5602	161.8283
479	5.75	DL+LL+WIND Full	Combination		-3959.125	-85.658	15.096	-28.841	408.094
479	0	DL+WIND Full	Combination		-4048.497	-100.9	18.25	70.0729	-97.6877
479	2.875	DL+WIND Full	Combination		-4095.464	-100.9	18.25	17.605	192.3999

479	5.75	DL+WIND Full	Combination		-4142.431	-100.9	18.25	-34.8629	482.4875
479	0	DL+LL+WIND Empty	Combination		-2664.92	-85.689	15.078	57.8913	-84.5591
479	2.875	DL+LL+WIND Empty	Combination		-2711.887	-85.689	15.078	14.5417	161.7963
479	5.75	DL+LL+WIND Empty	Combination		-2758.854	-85.689	15.078	-28.8078	408.1516
479	0	DL+WIND Empty	Combination		-2848.226	-100.931	18.232	70.0026	-97.8093
479	2.875	DL+WIND Empty	Combination		-2895.193	-100.931	18.232	17.5865	192.3679
479	5.75	DL+WIND Empty	Combination		-2942.16	-100.931	18.232	-34.8297	482.545
479	0	EQ Full	Combination	Max	-1535.791	55.249	58.656	97.5346	62.1847
479	2.875	EQ Full	Combination	Max	-1571.016	55.249	58.656	74.0904	99.2949
479	5.75	EQ Full	Combination	Max	-1606.241	55.249	58.656	248.7106	270.664
479	0	EQ Full	Combination	Min	-2719.683	-60.275	-61.558	-108.7016	-81.5265
479	2.875	EQ Full	Combination	Min	-2754.908	-60.275	-61.558	-76.9142	-104.1857
479	5.75	EQ Full	Combination	Min	-2790.133	-60.275	-61.558	-243.1911	-261.1039
479	0	EQ Half	Combination	Max	1006.208	196.223	29.462	-14.5821	181.8556
479	2.875	EQ Half	Combination	Max	970.983	196.223	29.462	45.9042	-186.3348
479	5.75	EQ Half	Combination	Max	935.758	196.223	29.462	304.4549	-420.2662
479	0	EQ Half	Combination	Min	-177.684	80.699	-90.751	-220.8183	38.1444
479	2.875	EQ Half	Combination	Min	-212.909	80.699	-90.751	-105.1004	-389.8154
479	5.75	EQ Half	Combination	Min	-248.134	80.699	-90.751	-187.4468	-952.0341

Table 9.2. Column Forces for label 475,477 and 479

9.2.2. Deformed shapes

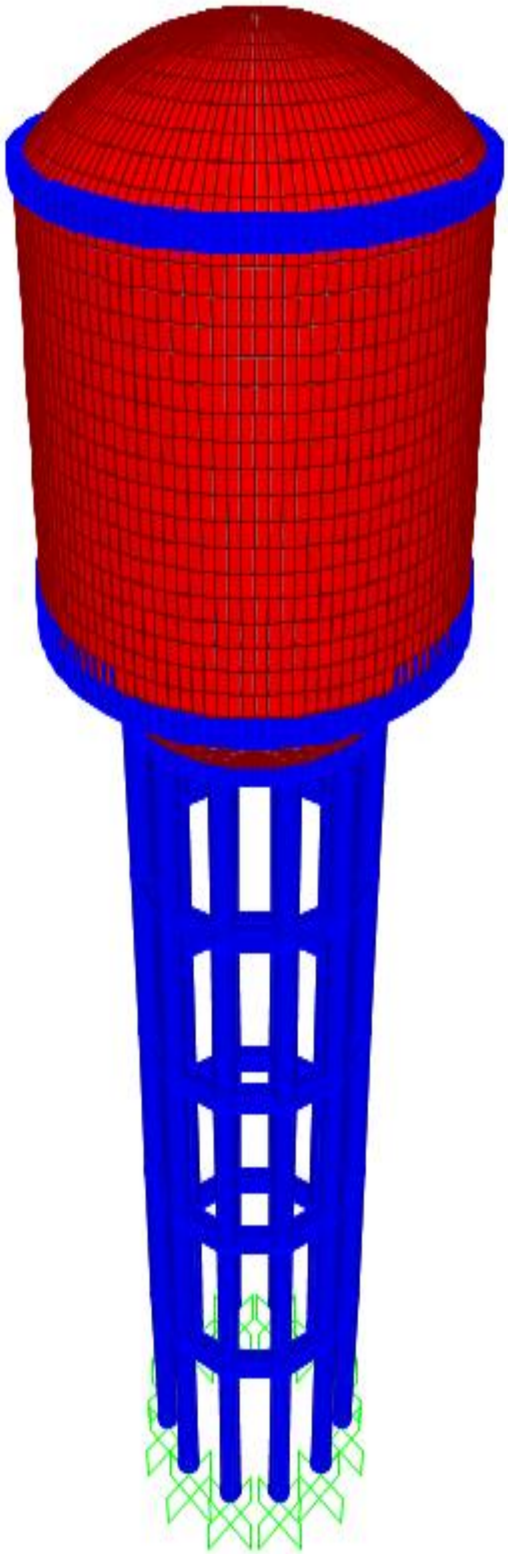


Fig.9.7. Un-deformed shape

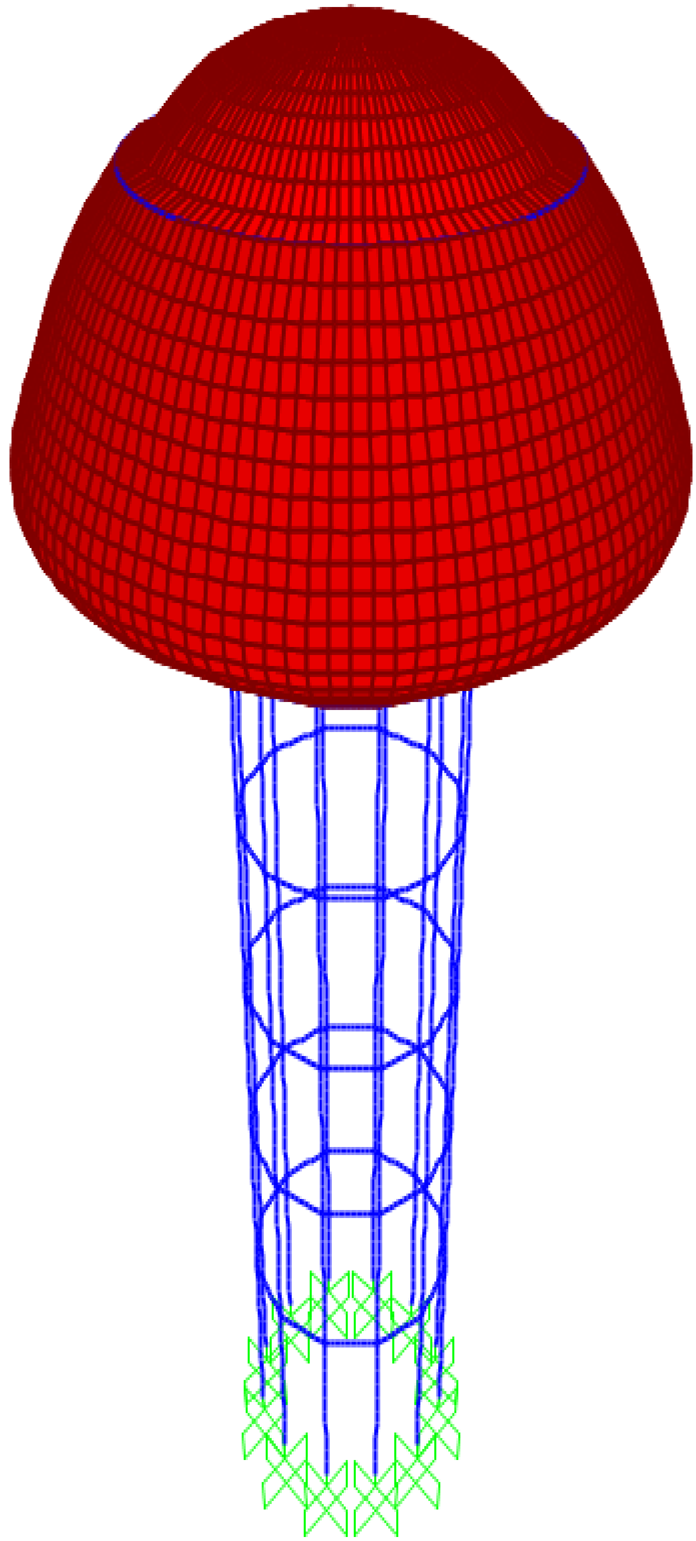


Fig.9.8. Deformation due to full lateral Water pressure

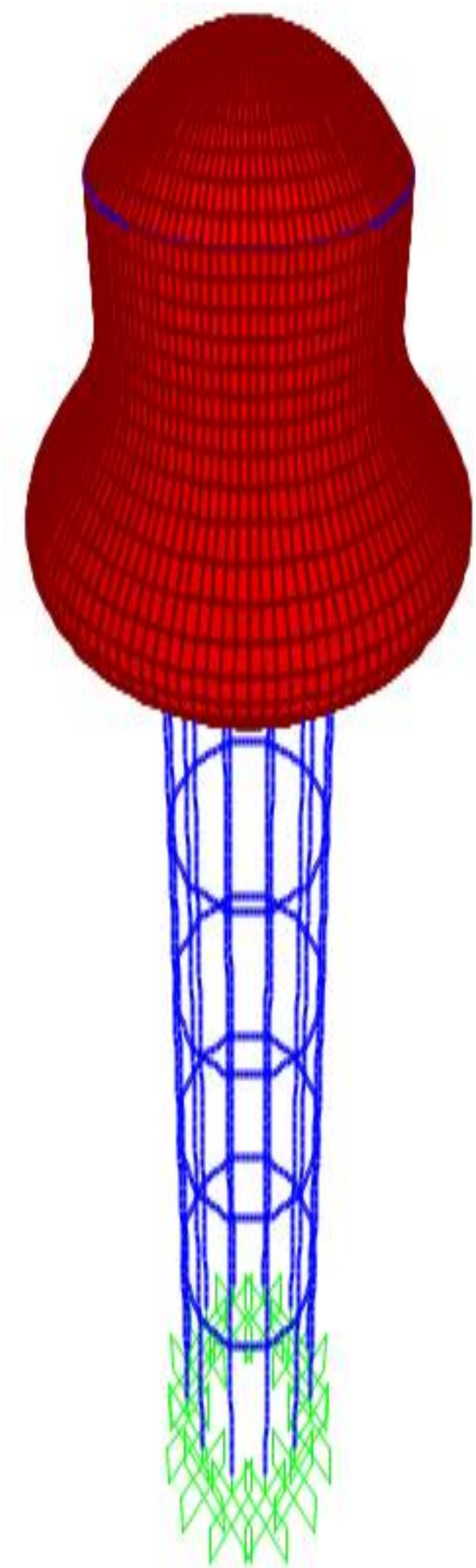


Fig.9.9.Deformation due to half lateral water pressure

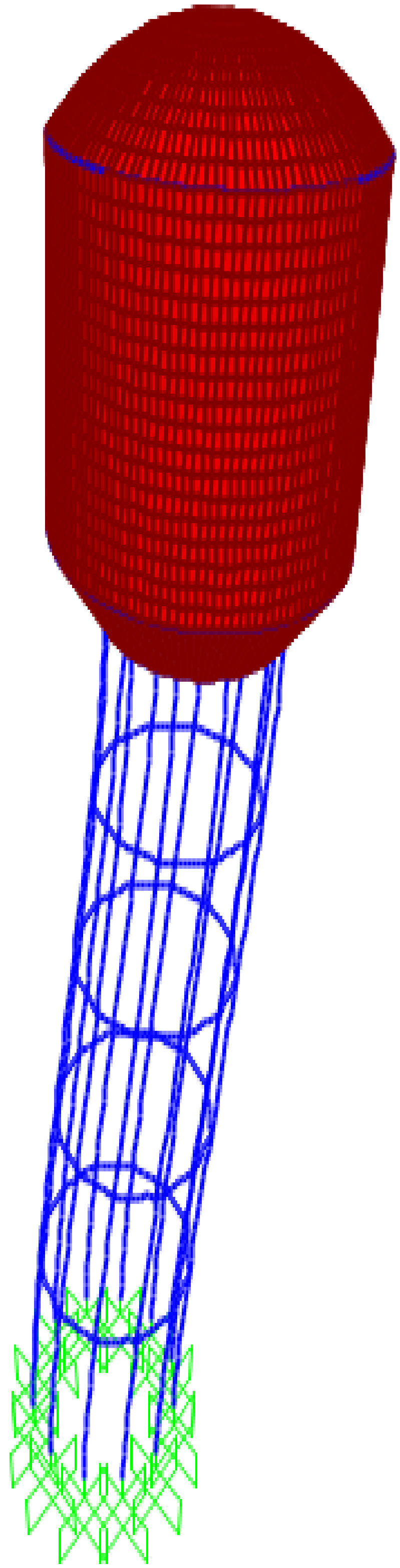


Fig.9.10.Deformation due to wind load

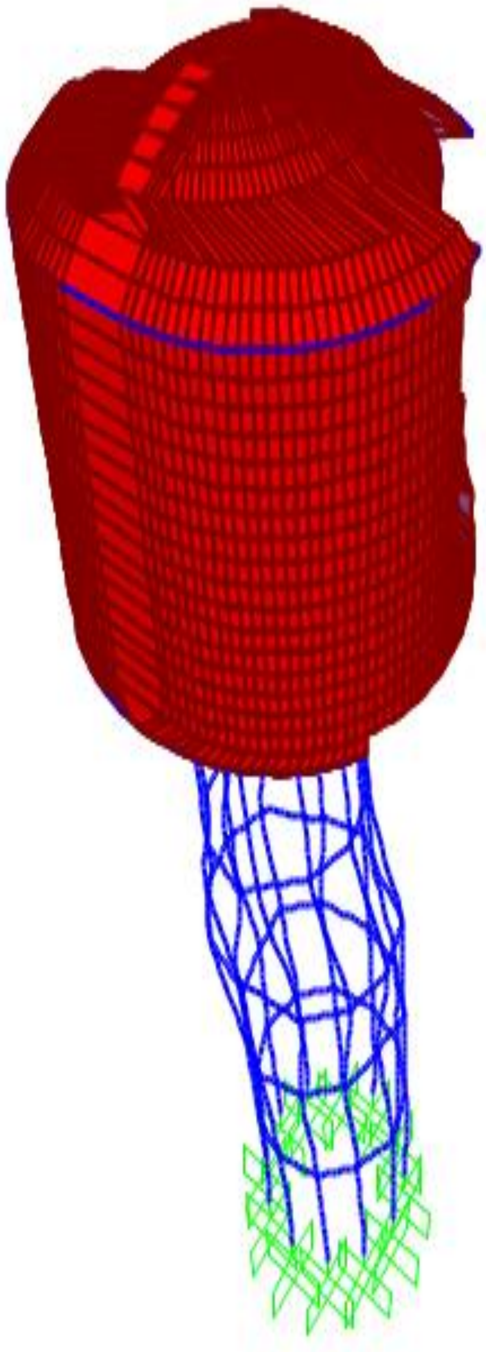


Fig.9.11.Deformation due to earthquake

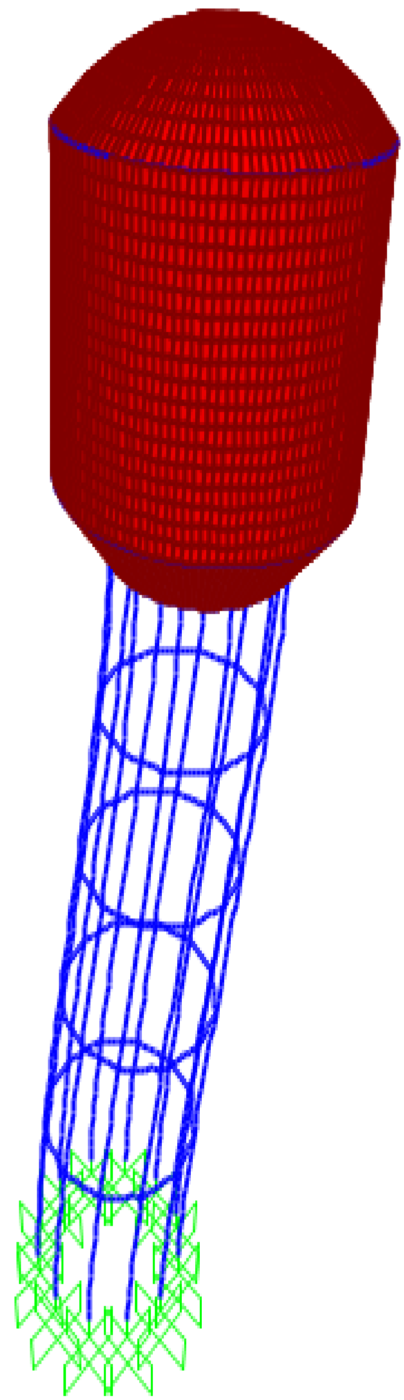


Fig.9.12.Deformation due to modal (mode-2)

In the deformed shape due to earthquake loading it clear that the contribution of torsional mode is very height. The first and second modes which are purely translational have comparatively close period of vibration to the torsional mode.

Chapter 10
Design of Column

10.0. Introduction

For the design of columns SAP2000 V18 is used. The design code used is Eurocode-2,2004 with appropriate NDP (national determined parameter). The table below shown the values used in the design of column.

	Item	Value
1	Design Code	Eurocode 2-2004
2	Country	CEN Default
3	Combinations Equation	Eq. 6.10
4	Reliability Class	Class 2
5	Second Order Method	Nominal Curvature
6	Multi-Response Case Design	Envelopes
7	Number of Interaction Curves	24
8	Number of Interaction Points	11
9	Consider Minimum Eccentricity?	Yes
10	Theta0 (ratio)	5.000E-03
11	GammaS (steel)	1.15
12	GammaC (concrete)	1.5
13	AlphaCC (compression)	0.85
14	AlphaCT (tension)	1.
15	AlphaLCC (lightweight compression)	0.85
16	AlphaLCT (lightweight tension)	0.85
17	Pattern Live Load Factor	1.
18	Utilization Factor Limit	0.95

Table 10.1. Design values

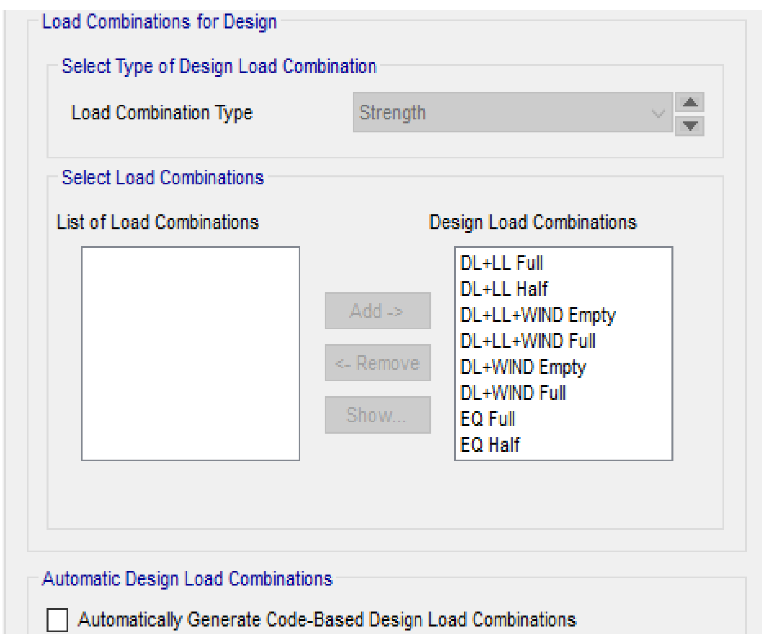


Table 10.2.Load combination

10.1. Design of column

10.1.1. Longitudinal design of column

Based on the Eurocode-2, 2004 the column is designed for both longitudinal and transverse reinforcement. In this subsection design of column for longitudinal reinforcement will be presented. The next table shows the longitudinal reinforcement required in mm^2.

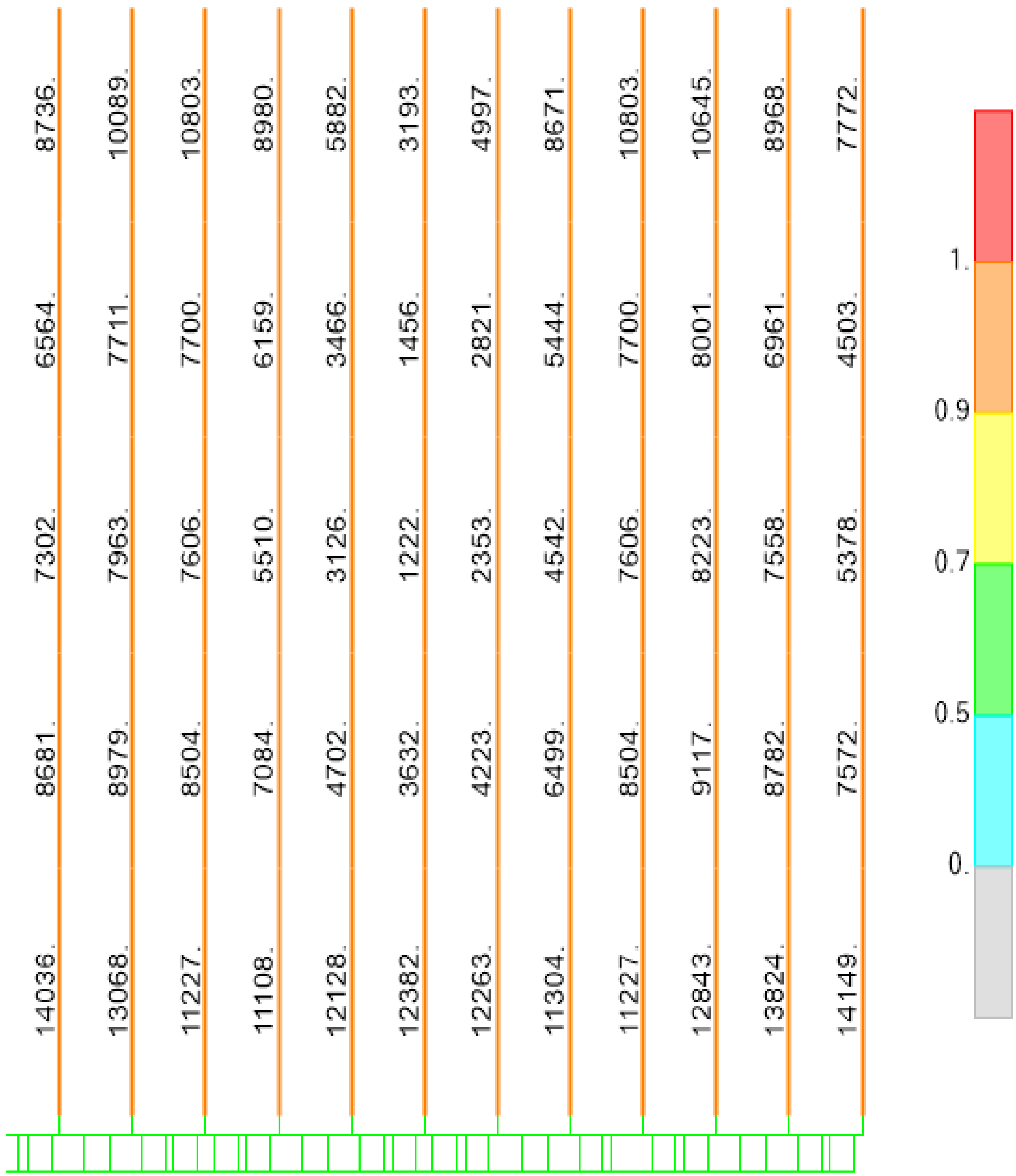


Fig.10.1. Longitudinal reinforcement

10.1.2. Transverse reinforcement

Also the shear reinforcement design is made based on the Eurocode-2, 2004 and the amount of reinforcement shown in the next figure. SAP2000 gives the shear reinforcement in terms of the area of shear reinforcement per spacing ($\frac{A_{sv}}{s}$).

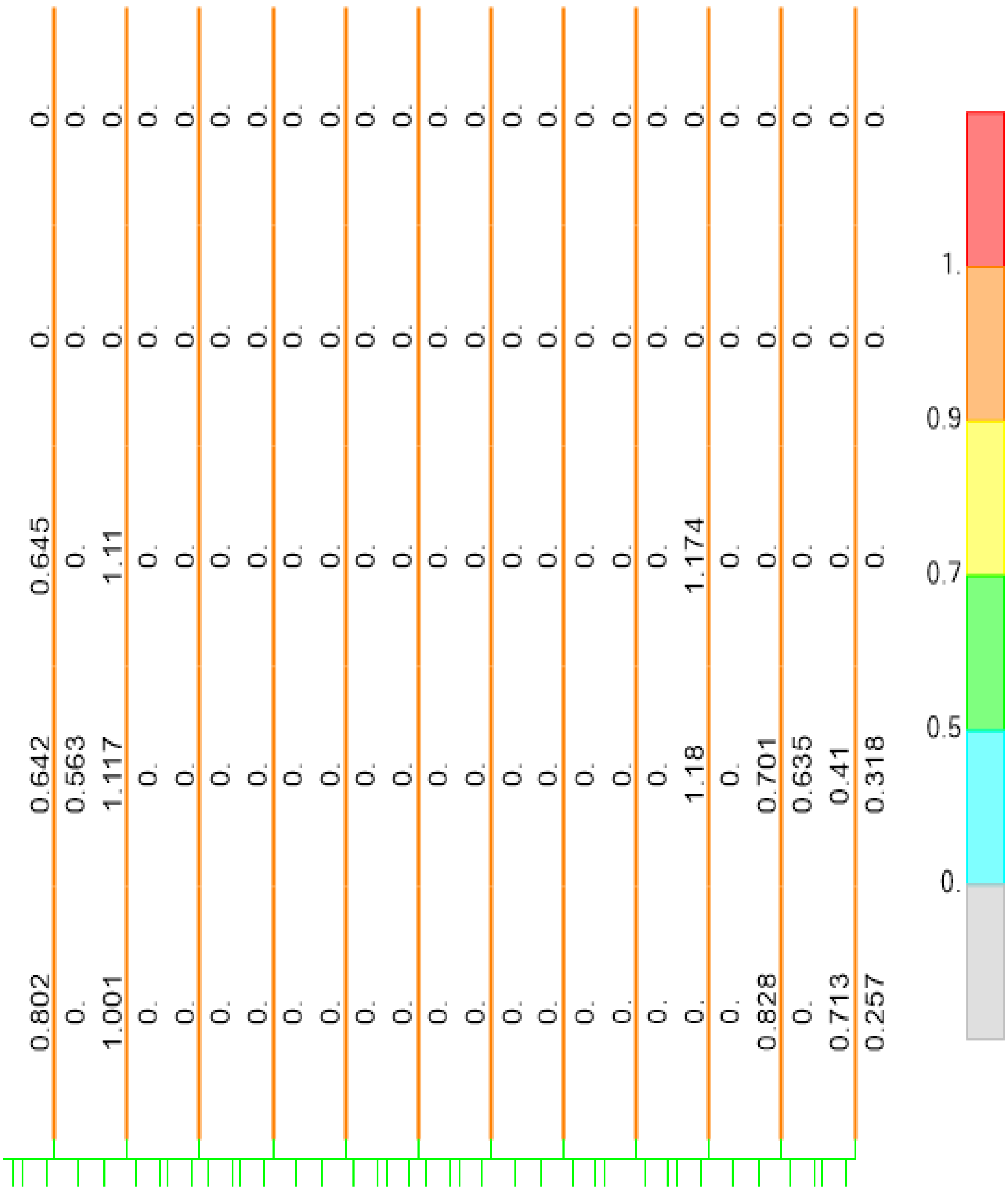
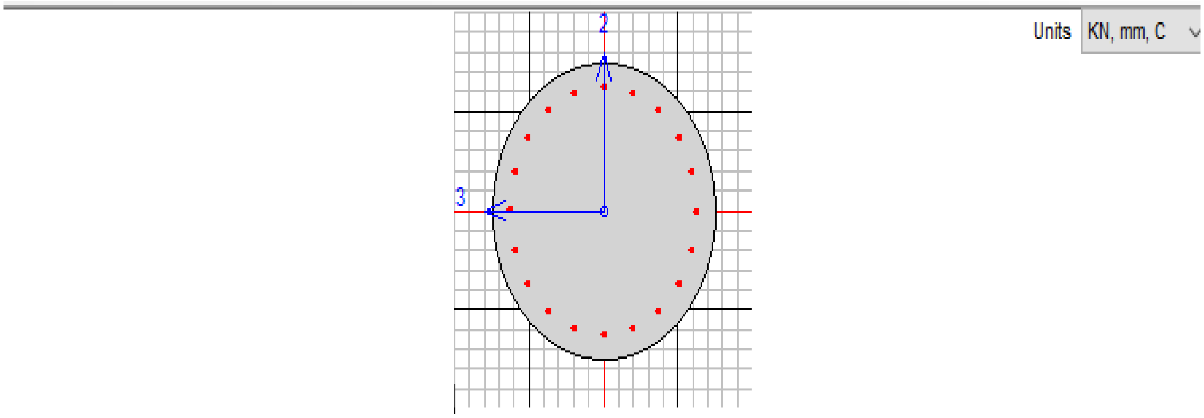


Fig.10.2. Shear reinforcement

For one column detail calculation of the design of column is shown in the next figure.




```
Eurocode 2-2004 COLUMN SECTION DESIGN  Type: DC LOW MRF  Units: KN, mm, C  (Summary)

L=5750.000
Element      : 481              D=800.000       dc=43.000
Section ID   : C-Col-800        E=29.000        fck,cyl=0.020   Lt.Wt. Fac.=1.000
Combo ID     : EQ Half          fyk=0.400       fywk=0.400
Station Loc  : 0.000            RLLF=1.000      SOM: Nominal Curvature
Combo Eq.    : Eq. 6.10

Gamma(Concrete): 1.500          AlphaCC=0.850   AlphaCT=1.000
Gamma(Steel)   : 1.150          AlphaLCC=0.850  AlphaLCT=0.850


AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR NEd, MEd2, MEd3
              Rebar      Design      Design      Design     Minimum     Minimum
               Area         NEd        MEd2        MEd3          M2          M3
          14149.104   -1456.082  248202.667 -900648.805       0.000       0.000

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                           M0Ed        Madd     Minimum        Beta           L
                         Moment      Moment         Ecc      Factor      Length
   Major Bending(M3) -535228.922      0.000       0.000       1.000    5750.000
   Minor Bending(M2)  189571.895      0.000       0.000       1.000    5750.000

SHEAR DESIGN FOR V2,V3
                          Rebar       Shear       Shear       Shear
                          Asw/s         VEd        VRdc        VRds
   Major Shear(V2)        0.713     168.994      60.065     168.994
   Minor Shear(V3)        0.257      60.831      60.065      60.831
```

AXIAL COMPRESSION RATIO

	Conc.Capa A*fcd	CompRatio Ned/(A*fcd)	CompRatio Limit	Seismic Load?	CompCheck Needed?	Ratio OK?
	5696.755	-0.002	0.550	Yes	No	Yes

JOINT SHEAR DESIGN

	Joint Shear Ash	Shear VEd,Top	Shear Vjhd	Shear Vrd,Conc	Shear Ratio	Joint Area
Major Shear(V2)	N/C	N/C	N/C	N/C	N/C	N/C
Minor Shear(V3)	N/C	N/C	N/C	N/C	N/C	N/C

(1.3) BEAM/COLUMN CAPACITY RATIOS

	Major Ratio	Minor Ratio
	N/A	N/A

Notes:

N/A: Not Applicable

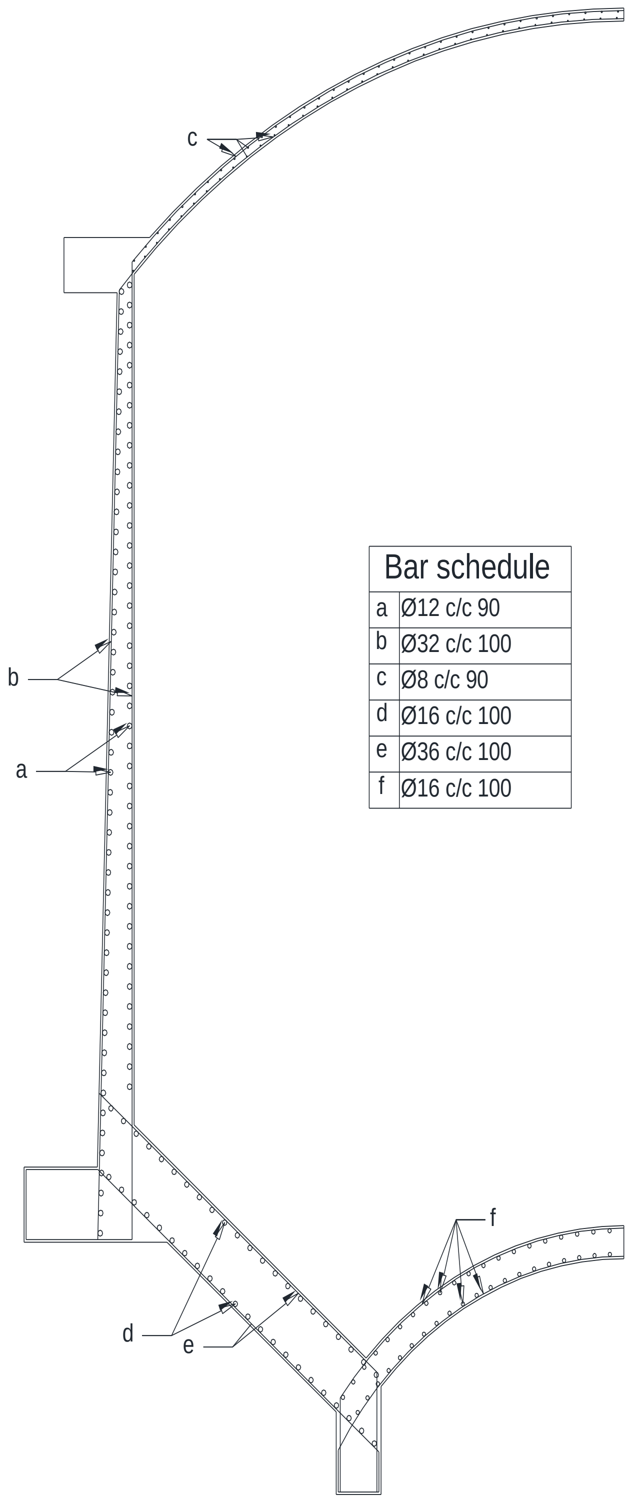
N/C: Not Calculated

N/N: Not Needed

Fig.10.3. Detail design calculation

Chapter 11
Detailing

11.0. Container tank detail



Bar schedule	
a	Ø12 c/c 90
b	Ø32 c/c 100
c	Ø8 c/c 90
d	Ø16 c/c 100
e	Ø36 c/c 100
f	Ø16 c/c 100

Fig.11.1. Container Detail

11.1. Column detail

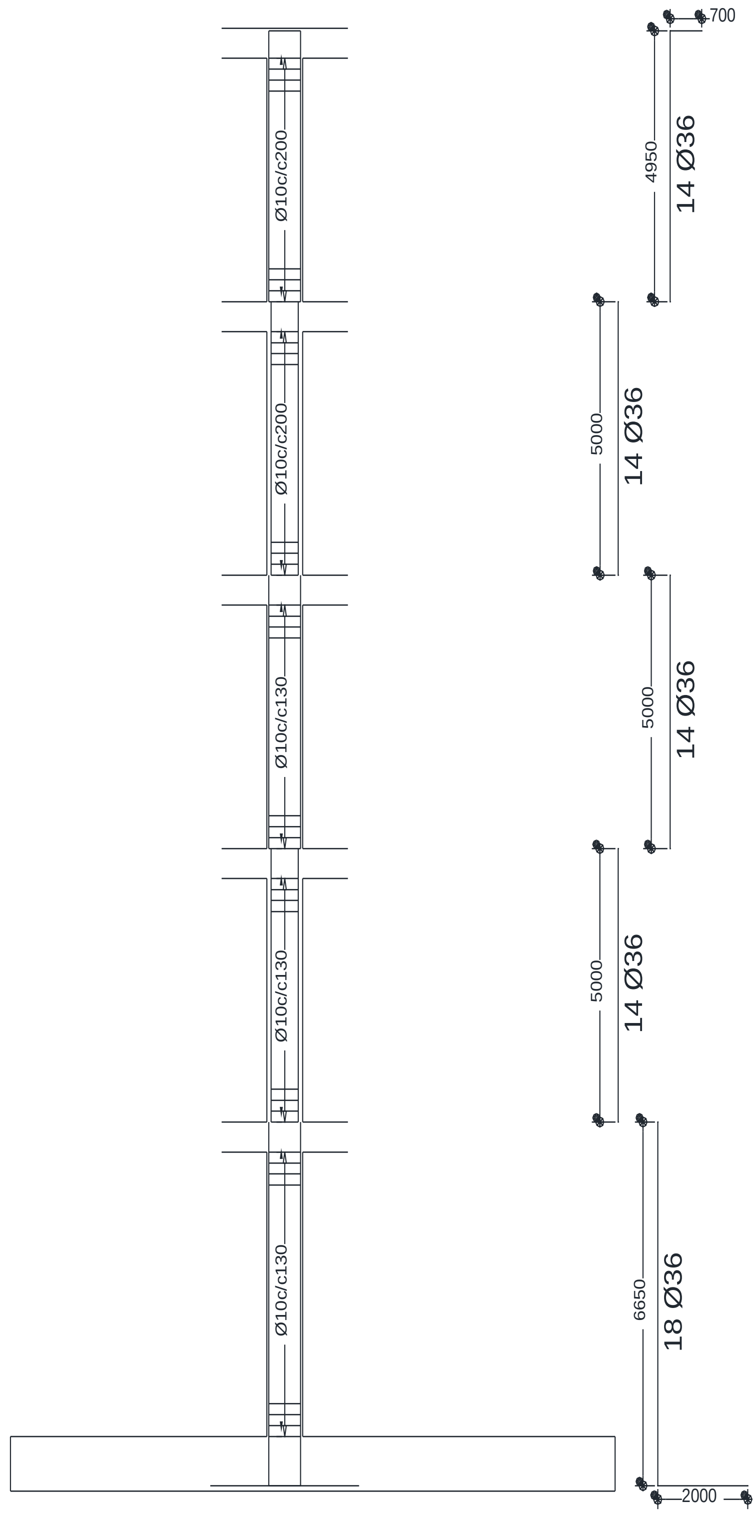


Fig.11.2. Column detail

Chapter 12
Foundation Design

12.0. Introduction

For the design of the foundation of the Intze water tank SAFE 2014 finite element software is used. The foundation type recommended is mat type with beams and having circular shape. The load on the foundation is taken from the analysis result of SAP 2000 software and applied on SAFE 2014. The amount of reinforcement required for the mat foundation as well as for the beam is taken out of the software. The assumed ultimate bearing capacity of the soil is taken to be 300kPa.

12.1. Modeling

The modeling of mat foundation along with the beams is shown in the next figure. The thickness of the mat is 800mm where the size of the beam is 400mm by 800mm where 800mm being the depth of the beam.

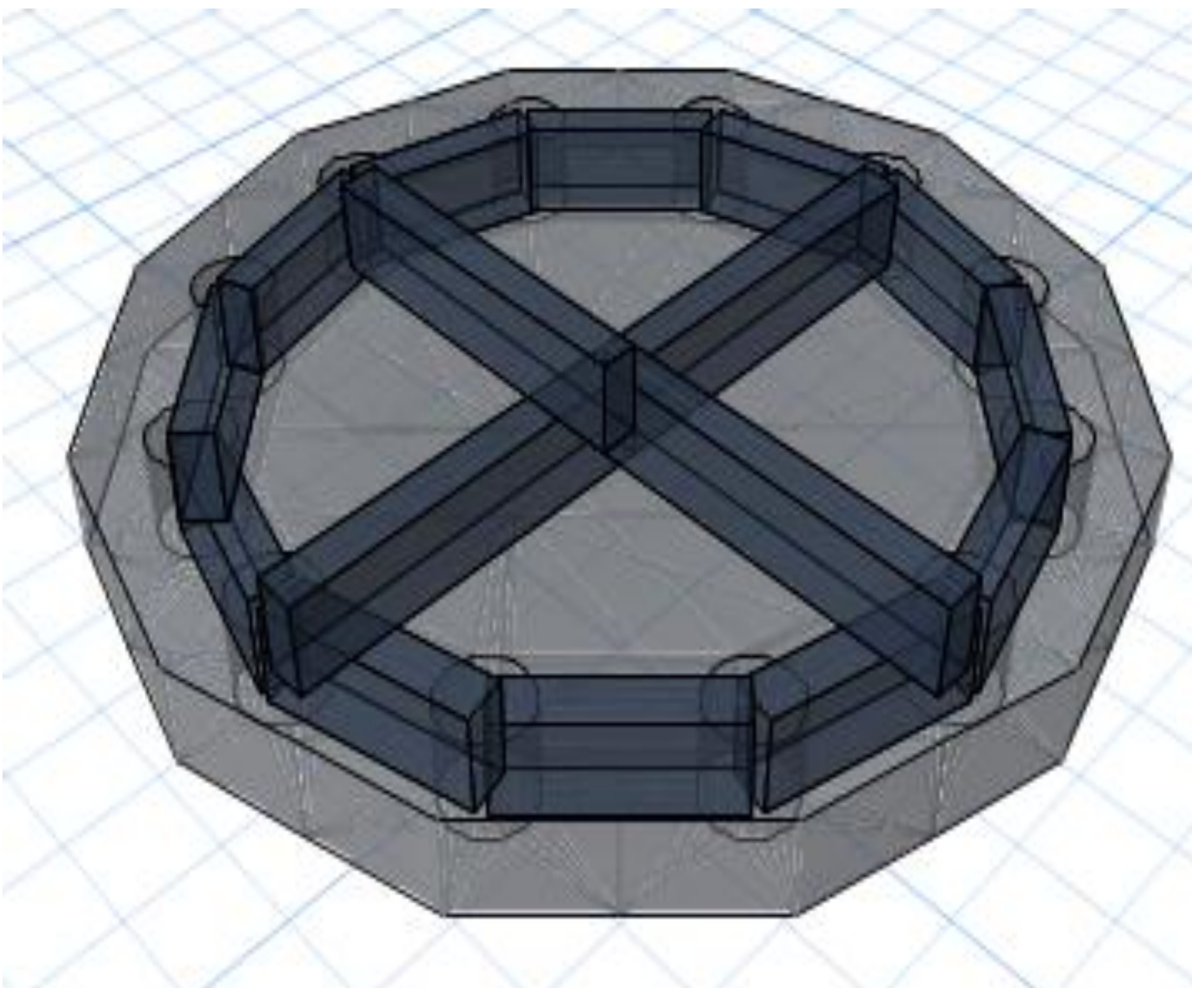


Fig.12.1. Foundation Model

At each joint the load is applied taken from SAP2000 output.

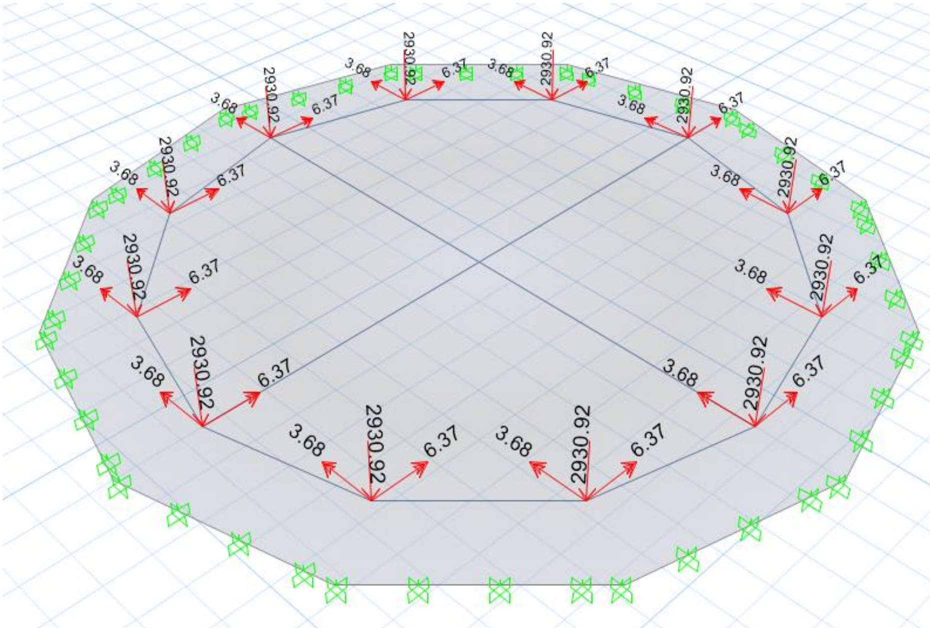


Fig.12.2. Loading Value

12.2. Result

12.2.1. Settlement

The settlement of the foundation under the design load is calculated from the software is shown in the next figure.

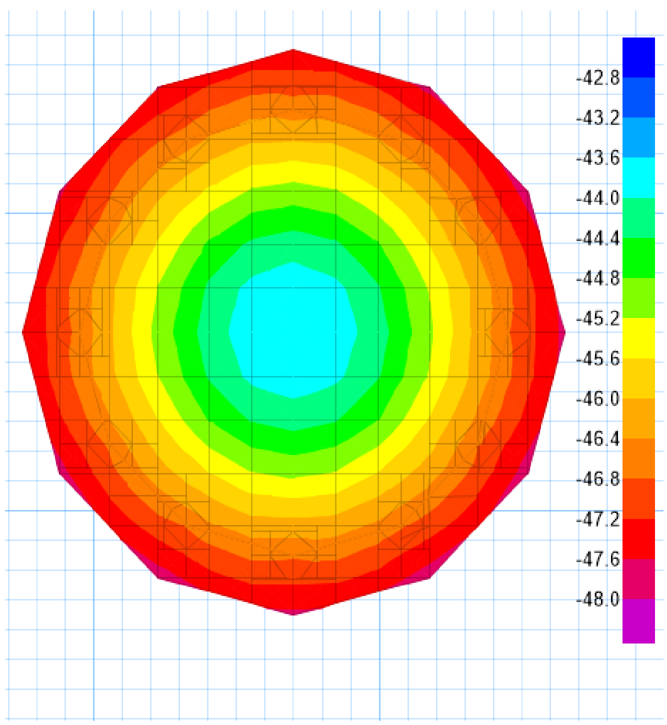


Fig.12.3. Settlement of foundation in mm

From the result it can be seen clearly that the distribution the settlement of the foundation is even being the difference between the largest and smallest settlement 6mm only.

12.2.2. Soil pressure distribution

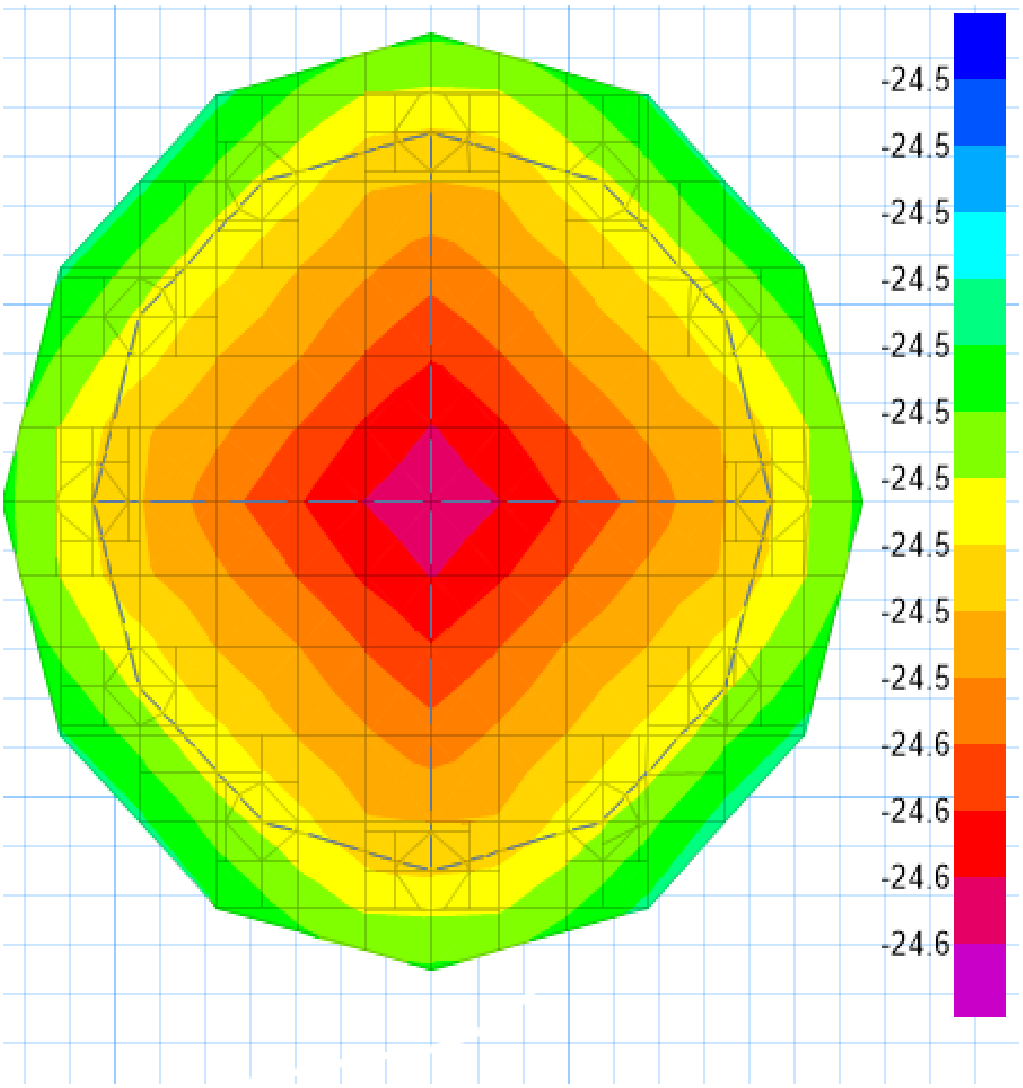


Fig.12.4. Soil pressure distribution

12.3. Design

The design of the foundation along with its beam is done using the finite element software. The results are displayed in the next figure. The amount of reinforcement can be converted to a spacing using the appropriate formula.

12.3.1. Mat reinforcement

The amount of mat reinforcement for both direction is given in below again for both top and bottom sides.

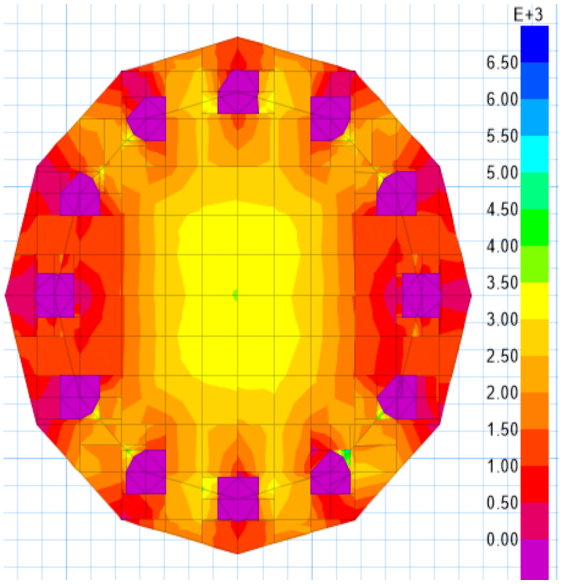


Fig.12.5.Reinforcement in X-direction (Bottom)

Use ø20 c/c 150mm

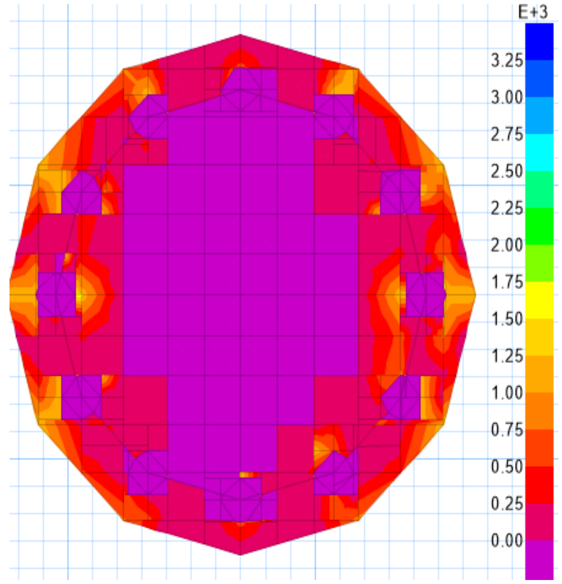


Fig.15.6.Reinforcement in X-direction (Top)

Use ø24 c/c 120mm

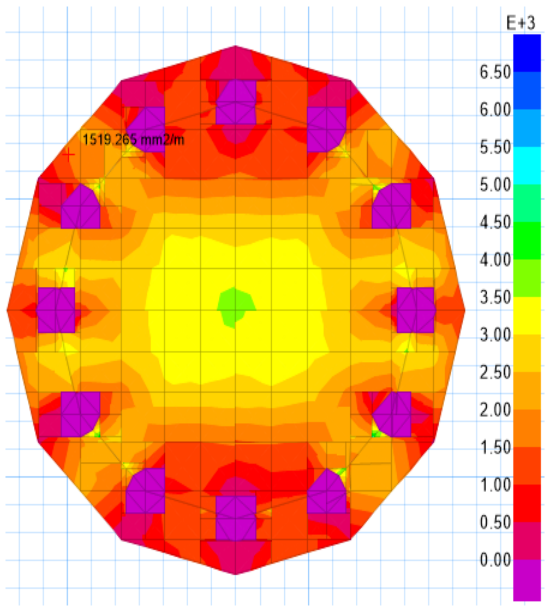


Fig.12.7. Reinforcement in Y-direction (Bottom)

Use ø20 c/c 150mm.

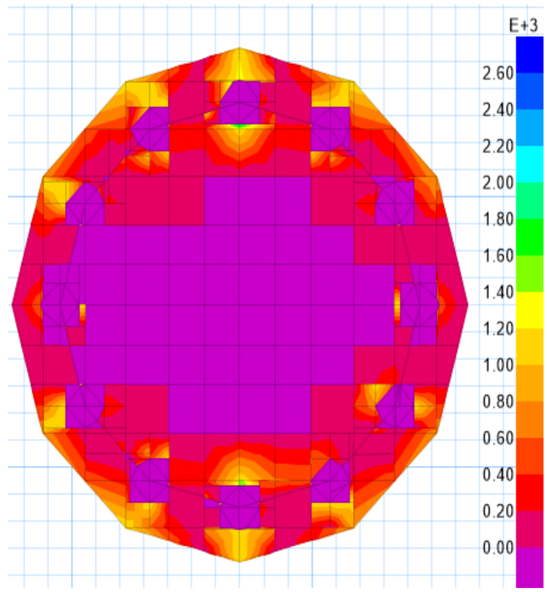


Fig.12.8.Reinforcement in Y-direction (Top)

Use ø20 c/c 120mm.

12.3.2 Beam Design

The beam is designed for both flexure and shear in SAFE 2014. The flexural reinforcement is given in terms of mm^2 and the shear reinforcement is given as ($\frac{A_{sv}}{s}$). The next two figures will provide the amount of reinforcement required for the mat and beam.

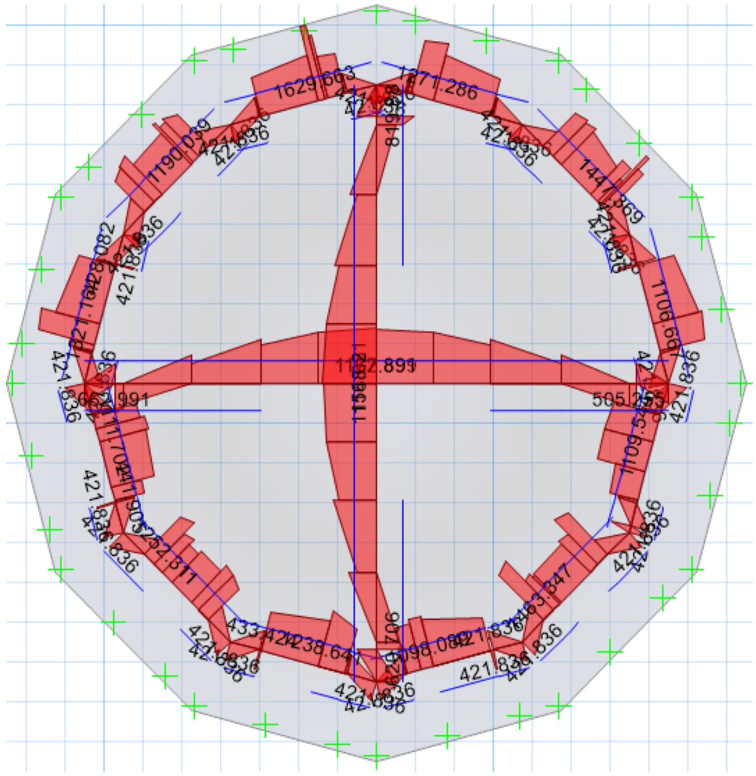


Fig.12.9. Flexural reinforcement of beam

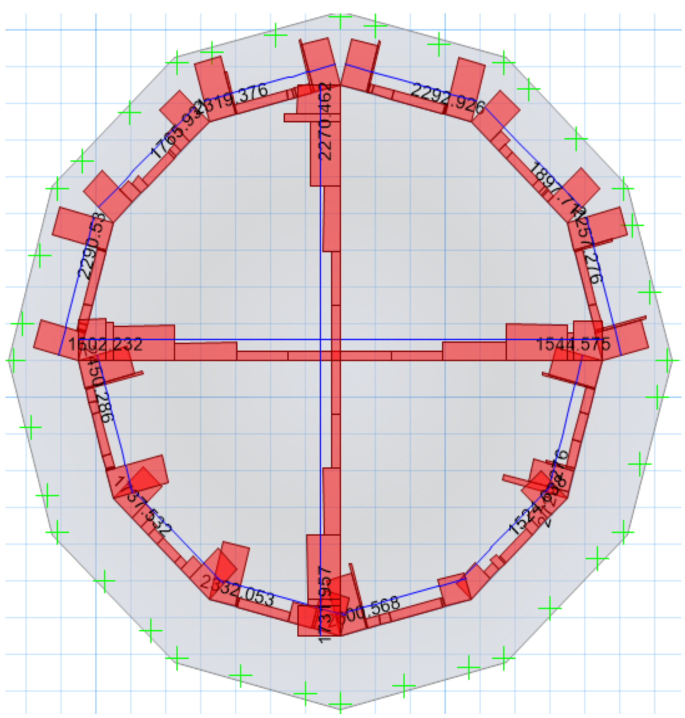


Fig.12.10. Shear reinforcement of the beam

Discussion:

1) The ring beam will be subjected to zero hoop stress when the tank is full. Horizontal thrust is taken by a rib encircling the edge of the roof dome is provided .A ring beam is provided to transmit the load to the columns which is provided at the junction of domed bottom and conical potion.
2) Sloshing wave height is assumed small, it results additional hydrodynamic pressure. Sloshing is defined as the periodic motion free liquid surface in partially filled containers. It is caused by any disturbance to partially filled liquid containers.
3) Intze tanks are extensively used for storing water for civic purposes because of their optimal load balancing shape.
4) Hoop tensile force will mostly govern the thickness of conical and cylindrical walls. The thickness of spherical bottom dome will be governed by the maximum compressive stress of the meridonial compressive force and bending moment at the edge.
5) Due to many degrees of redundancy, Stress analysis of Intze tanks is extremely complicated however with certain approximations the elastic theory of thin shells were used to analyze these tanks with sufficient accuracy.
6) The principle stress system had obtained using the membrane theory of shells. Stresses in an Intze type tank due to primary loading using elastic theory. Secondary stresses due to shrinkage, temp variation and wind forces should also calculate for critical designs.
7) In the Intze tank the various components of the tanks shall be checked from the various perspectives. Let us start with the dome which is usually provided with thickness of 100 to 150mm and reinforcement must be laid along meridonially and latitudinal. Here the hoop stress is less than 1.5 KN/m2. Therefore the top dome for the thickness of 150 mm minimum reinforcement must be provided. Ring beam which supporting the dome is necessary for resisting the horizontal thrust developed by the dome, therefore this beam is to be designed from the hoop tension. Below this ring beam cylindrical walls are there, they should be designed for hoop tension caused by the water pressure. In the flow of loads the next is the ring beam at the junction of cylindrical walls and the conical wall; therefore it is to be designed for hoop tension. Basically it provide resistant to the horizontal component of the reaction of the conical wall on the cylindrical wall. Here larger width of the beam will serve the purpose of walk way around the tank. It is the conical slab in next which is also to be designed from the hoop tension point of view. Basically it is to be designed as slab which is spanning ring girder at the bottom and the ring beam at the top. The last part in the tank system which is connecting the tank portion with the staging part that is ring girder that means it is supporting all the tank and its components. Finally it is resting on the columns therefore it is to be designed from bending moment and torsion point of view. Columns which transfer the load are to design from the gravity loads and wind load.

8) Based on the type of force acting in the member, the member should satisfy the various types of requirements.

   a) Members subjected to axial tension only: In this condition the member should satisfy the requirements. There should be sufficient reinforcement to resist all the tensile force. Assuming that the concrete is un cracked and reinforcement act together to resist the direct force, the calculated tensile stress in concrete should not exceed the maximum permissible stress in concrete in direct tension.

   b) Members subjected to bending moment only: Neglecting the concrete in tension zone, the compressive stress in concrete should not exceed the permissible value and tensile stress in steel should not exceed the permissible values. Assuming concrete to be uncracked the tensile stress in concrete should not exceed the permissible tensile stress in bending. For cracked condition the usual procedure of designing singly reinforced beam (or doubly reinforced beam if required) will be followed here but with the reduced stresses in steel reinforcement. For un cracked condition, in this case assume that the whole section is resisting the moment and calculate the maximum tensile stress in concrete which should not be more than permissible value.

   c) Members subjected to combined axial tension and bending: For the members subjected to combined axial tension and bending moment. It requires for no crack condition that the stresses due to combination of direct tension and bending moment shall satisfy the following condition.

$$\frac{f_{ct}}{\sigma_{ct}} + \frac{f_{cbt}}{\sigma_{cbt}} \leq 1$$

f_{ct} –Calculated direct tensile stress in concrete
σ_{ct}- Permissible direct tensile stress in concrete
f_{cbt} –Calculated stress in concrete in bending tension
σ_{cbt} – Permissible stress in concrete in bending tension

Conclusion:

1) About 70% of liquid mass is excited in impulsive mode while 29.5% liquid mass participates in convective mode. Sum of impulsive and convective mass is 1136410 kg which is about the total mass of liquid in the earth quake analysis.
2) Finally the earth quake forces are governing the design of the elevated water tank.

Future scope of the work:

i) The Intze tank cost estimation can be done

Appendix-A

Moment coefficients in circular girders supported on columns:

No of columns	**Positive bending moment at center of spans**	**Negative B.M at support**	**Maximum twisting moment**	**Angle between columns**	**Angular distance for maximum torsion**
N	K_2	K_1	K_3	Degrees	Degrees
4	0.0176	0.0342	0.0053	$19^0 22'$	90
6	0.0075	0.0142	0.0015	$12^0 44'$	60
8	0.0041	0.0083	0.0006	$9^0 33'$	45
10	0.0023	0.0054	0.0003	$7^0 30'$	36
12	0.0014	0.0037	0.00017	$6^0 15'$	30

Table13.0. Moment coefficients

Source: Ramamrutham.S, 1978, design of Reinforced concrete structures, 8th edition, DanpathiRai publications.

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